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CONTENTS

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover

LATERAL EARTH PRESSURE: THE ACCURATE EXPERIMENTAL DETERMINATION OF THE LATERAL EARTH PRESSURE, TOGETHER WITH A RESUME OF PREVIOUS EXPERIMENTS

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To be Presented March 7, 1923.

Synopsis

This paper describes the first complete set of tests using large-sized apparatus for measuring directly by means of platform scales the components of the active pressure on a model wall, 6 ft. high and 5 ft. wide. The only restraints on the wall were the measuring devices. Tests with sand under various conditions, using vertical and battered walls backed with wood, glass, and sheet iron, indicate the reliability of the Poncelet theory of the maximum wedge causing oblique pressure. Tests with walls battered in both directions show that the direction of the resultant pressure is always inclined to the normal at an angle equal to the angle of friction on the back of the wall. The effect of surcharge does not agree with the ordinary theoretical assumptions. Settling and variations in temperature cause remarkable changes in the lateral pressure.

The discussion of recent experimental data, together with a brief outline of the earlier experiments, furnishes only the minimum information necessary for a comparison between the former and the writer's results. The methods of analyzing the earth-pressure problem with the restrictions on each method, are the result of a study of more than 650 references, covering practically everything written on the subject of earth pressure. The suggested practical formulas for the determination of the lateral pressure in retaining wall design are based on the experimental data.

Appreciation

The experimental determination of lateral earth pressure was assigned to the writer when he was appointed a Baldwin Fellow in Civil Engineering at the University of Cincinnati, during July, 1919. The problem has been studied by the Civil Engineering Department of the University since 1912, when G. M. Braune, M. Am. Soc. C. E., then Professor in Civil Engineering, conceived the idea of a large size apparatus to test the lateral pressure of soils. With the co-operation of Professor C. C. Meyers, of the Department of Industrial Engineering, a preliminary design was made, based on that of Mueller-Breslau.† During 1916, B. H. Wolfkoetter, Fellow in Civil Engineer-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited, and may be sent by mail to the Secretary. Discussion on the paper will be closed in **August, 1923**, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

* Brooklyn, N. Y.

† H. Mueller-Breslau, "Erddruck auf Stützmauern" (1906).

ing, developed the details for this design and the writer, under the advisership of Professor Braune, completed the present design. Suggestions were made by several men, who are mentioned in the description of the apparatus. To Professor Braune is also due the credit of completing the apparatus, obtaining the necessary funds, etc. E. K. Ruth, M. Am. Soc. C. E., checked the plans and computations of the design, Mr. Gerald Fitzgerald aided the writer in assembling the apparatus and in the preliminary tests. The Stacey Bros. Gas Construction Co., of Elmwood, Ohio, furnished the measuring parts of the apparatus.

The writer wishes to express his gratitude to Professor Braune, who, as adviser in 1919-21, interested him in the subject and supervised the work of designing, constructing, and erecting the apparatus. He is also indebted to his later adviser, E. D. Gilman, Assoc. M. Am. Soc. C. E., who has made many suggestions that have aided in bringing the experimental and theoretical parts of this work to a successful end. Thanks are also due to Professor L. T. More, Dean of the Graduate School of the University of Cincinnati, and D. B. Steinman and F. E. Schmitt, Members, Am. Soc. C. E., for advice and encouragement, without which the writer would probably not have completed the work.

Introduction

Study of the earth-pressure problem has been greatly hindered by lack of uniformity in notation and in definitions. It is hoped that the following definitions of the factors entering the determination of the lateral pressure of granular materials, will lead to standardization. As far as possible, reference is made to the originator of each word or idea defined. The necessity for such detail in definition will be understood from a comparison of the standard books on the subject. There seems to be a hopeless confusion of nomenclature as well as a lack of information on the history and development of the theories. The following notation will be used:

Distances:

b = width of wall, measured horizontally.
h = height of fill, measured vertically.
l = length of wall, measured along the back.
s = height of equivalent fill, in case of surcharge.
x = height of resultant above base of wall.

Densities:

w = weight of wall, in pounds per cubic foot.
y = weight of fill, in pounds per cubic foot.

Coefficients:

c = coefficient of cohesion, in pounds per square foot.
f = " of friction, corresponding to natural slope.
f' = " of friction of the fill on wall.
f_1 = " of internal resistance.
f_2 = " of internal friction.

Forces:

$E =$ total theoretical earth pressure.

$P =$ resultant experimental earth pressure, the components being, H, horizontal; V, vertical; N, normal; and T, tangential.

Angles:

$\alpha =$ angle between the vertical through the toe and the back of the wall and is negative if the top of the wall overhangs the toe.

$\delta =$ angle between the resultant earth pressure and the horizontal.

$\phi =$ angle of natural slope or repose.

$\phi_1 =$ angle of internal resistance.

$\phi' =$ angle of friction between wall and fill.

$\epsilon =$ angle between the free earth surface and the horizontal and is negative if the surface is below the horizontal.

$\omega =$ angle between the plane of rupture and the vertical.

Coefficients:

C : of $E = \frac{1}{2} y h^2 C$, in the Poncelet theory.

$$C = \left[\frac{\cos(\phi - \alpha)}{(1 + n)\cos\alpha}\right]^2 \frac{1}{\cos(\phi' + \alpha)}$$

$$n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi - \varepsilon)}{\cos(\phi' + \alpha)\cos(\alpha - \varepsilon)}};$$

N : of $E = \frac{1}{2} y h^2 N$, in the restricted wedge theory (E normal).

$$N = \left[\frac{\cos(\phi - \alpha)}{(1 + n)\cos\alpha}\right]^2 \frac{1}{\cos\alpha};$$

$$n = \sqrt{\frac{\sin\phi\sin(\phi - \varepsilon)}{\cos\alpha\cos(\alpha - \varepsilon)}};$$

R : of $E = \frac{1}{2} y h^2 R$, in the Rankine theory.

$$R = \cos\varepsilon \frac{\cos\varepsilon - \sqrt{\cos^2\varepsilon - \cos^2\phi}}{\cos\varepsilon + \sqrt{\cos^2\varepsilon - \cos^2\phi}}$$

Natural Repose.—Any granular, semi-solid, or semi-fluid material, if unsupported, assumes a surface sloped at an angle to the horizontal; the maximum angle that a material can assume is called its natural slope, or angle of repose, which is designated by the symbol ϕ. The natural repose of liquids is horizontal. The resistance of the grains or particles of granular materials to rolling over each other tends to neutralize the effect of gravity in bringing the material to a level. At angles less than the angle of repose, a grain is held by this resistance, whereas, at angles more than the angle of repose, the resistance is less than the gravitational attraction and the particle

rolls down. Woltmann, in 1799, introduced the theory that the tangent of the angle of repose must be the coefficient of friction of the granular material on itself, and, until recently, this theory has been accepted. As the angle of repose is a surface phenomenon, a plane of equilibrium on the surface, one is not justified in assuming the same amount of resistance inside the mass. Experiments have proved that the angle of friction inside the mass is not the angle of repose.

Internal Resistance.—The coefficient of friction is determined experimentally by measuring the force required to move a known mass of material resting on a plane of the same material. It is found that two coefficients may be determined, one corresponding to the force required to start the mass in motion, the second, corresponding to the force required to keep the mass moving uniformly. In the first case, the force required is equal to the total resistance exerted by the mass to shearing along the plane. This resistance consists of two parts: The resistance of the particles to rolling over each other, called friction, and the surface attraction between particles, called cohesion. This coefficient is called the "coefficient of internal resistance", and the corresponding angle is the angle of internal resistance. When motion occurs, the friction acts, but the cohesion does not, so that the second coefficient found, which is smaller than the first, is called the "coefficient of internal friction", and the corresponding angle is the angle of internal friction.

Internal Friction.—It was pointed out by Boussinesq in 1883 that the formulas for the phenomena in a granular mass should be functions of the angle of internal friction rather than the angle of repose. In such phenomena, the mass is originally in a state of rest and changes to a state of non-equilibrium, either motion or unbalanced stress, whereas the angle of repose is measured after the granular material has changed from a state of non-equilibrium to a state of rest. This prediction has been fully realized in recent experiments.

Laws of Friction and Cohesion.—In 1781, Coulomb formulated the laws of friction of solid bodies and of the cohesion of bodies. He assumed that the same laws hold true for granular materials and with only few exceptions, all writers on the subject have made this assumption.

1.—The frictional resistance on any surface is equal to a constant, that is, the coefficient of friction, times the total normal pressure on that surface.

2.—The cohesion resistance is equal to a constant times the area of the surface.

Both forces act on the surface and in a direction opposite to that of the attempted motion. Whether the cohesion resistance is a cohesion, a surface attraction between like particles, or an adhesion, a surface attraction between unlike particles, is a doubtful point. It is probably an adhesion, because each grain is coated more or less by a film of water. In dry sand, there is no cohesion force; the attraction is then between the grains of earth and the water.

Active and Passive Pressures.—If a mass of earth is assumed to be divided into two sections by a plane, the forces exerted by one mass are just neutralized by the forces exerted by the other mass. Otherwise, the plane would not be in

equilibrium. Either section may be removed and replaced by a single resultant force without changing the state of equilibrium. The minimum value of this resultant force is the required strength of a wall that is to hold in place the remaining section. If this force, which is called the "active earth pressure", is increased, it will not only hold the earth in equilibrium but will also tend to move it upward. Such motion is resisted by the friction and cohesion in the mass. When the force has been increased to a value that just overcomes these resistances, and the earth is on the point of moving upward, it is called the "passive earth pressure". A better name for it would be the "passive resistance to pressure" because it is equal to the latent resistance which the mass can exert to overcome forces tending to cause upward motion. A retaining wall furnishes an example of active pressure, usually called "lateral earth pressure". Foundations furnish examples of passive pressure; when a heavy load causes the adjacent soil to heave, the passive resistance of the soil has been exceeded.

Determination of the Lateral Pressure.—Three factors are necessary for the complete determination of the lateral pressure, namely, magnitude, direction, and point of application. The theories of lateral pressure cannot be based on the laws of solid bodies, nor on the laws of fluids. A granular material is neither a solid nor a liquid; it is usually a combination of the two. The laws of granular materials have not been fully developed. All theories include some common assumptions:

1.—That the mass is homogenous.

2.—That the material consists of grains, which have the resistance to rolling over each other, called friction.

3.—That the laws of friction hold true.

4.—That the laws of cohesion hold true. Cohesion is usually disregarded, as it cannot be relied on to act at all times.

Wedge of Rupture.—If a retaining wall were removed, some of the retained fill would immediately slip down. Not all the material above the plane of repose would fall at the same time. The first slip is called the wedge or prism of rupture. The surface that is left is fairly plane and is usually called the plane of rupture. Weathering and shock will soon cause the material to assume its natural slope. It is true that clay banks with overhanging tops, ditches with vertical sides, etc., are often seen, but they are merely examples in which the cohesion is acting. Such banks and walls of ditches will soon slip and assume sloping sides unless retained.

Coulomb and Wedge Theories.—The early wedge theories, derived by Coulomb, Mayniel, and Prony assume that the pressure behind a wall is caused by the wedge of rupture exerting a normal pressure on the wall. The resultant was assumed to act at the one-third point of the height of the wall. In 1840, Poncelet developed the general wedge theory, in which the resultant pressure was assumed to act at the angle of friction between the fill and the wall from the normal to the wall. In all the wedge theories, an expression for the lateral pressure is derived in terms of the wedge, and a maximum value obtained by setting the first derivative of the expression equal to zero. Coulomb, in 1773, assumed the width of the wedge as the variable with respect to

which the differentiation is performed; all the other theories assume the wedge angle, that is, the angle between the wall and the plane of rupture, as the variable.

Rankine and Stress Theories.—By assuming the mass indefinite in extent and incompressible, the theory of the elasticity of materials can be applied to a unit parallelopiped of volume in the retained mass of earth. In this manner, Rankine obtains his theory. Levy and Boussinesq generalized the stress theory by taking into account the effect of the wall surface, calling it a restraint that causes a discontinuity in the mass.

Many minor theories have been issued, but lack of space prevents even an outline of them in this paper.

Design of the Apparatus

The design of the large scale apparatus used in the experiments was based on that of Mueller-Breslau in 1906, in that the same types of wall and bin were used. The readings, however, were to be obtained by means of scales. The plans were submitted for correction and criticism to a number of well known engineers. The plans required a steel bin, 5 ft. high with side-walls that could move parallel to the test wall, thereby attempting to eliminate the side-wall effect. Professor J. H. Smith, of the University of Pittsburgh, objected to this part of the design. Careful analysis by Professor Braune and the writer disclosed the error in this method of trying to eliminate the effect of the side walls because any motion of the side-walls, caused by forces existing in the fill, changes the volume and shape of the fill and prevents accurate determination of the existing pressures. William Cain, M. Am. Soc. C. E., warned against too weak a test wall, as deflections of the wall will cause a dissipation of the forces in the fill. The original design called for a thin steel wall, which was to be counterbalanced vertically. D. B. Steinman, M. Am. Soc. C. E., suggested that the test wall be left entirely free, as a retaining wall is not counterbalanced, but rests on the foundation. He also suggested that the effect of different methods of placing the fill behind the wall be determined. Allen Hazen, M. Am. Soc. C. E., suggested a careful study of the effect of vibration and shock, as well as the question of packing and water content. Robert A. Cummings, M. Am. Soc. C. E., offered the aid of the Special Committee on the Bearing Value of Soils for Foundations, etc., in the proposed investigation, and suggested co-operation to prevent unnecessary duplication of work. A. T. Goldbeck, Assoc. M. Am. Soc. C. E., showed the writer the methods used by the U. S. Bureau of Public Roads in conducting large scale tests in highway research, and gave some valuable suggestions for the actual testing.

The apparatus, as constructed, has proved entirely satisfactory, and it has proved to be accurate, quite easy to manipulate and sufficiently flexible that changes for the various tests could easily be made.

The following points were considered in the final design as necessary for the accurate determination of the lateral pressures of granular materials. (Fig. 1.) The pressures are measured on a "free" wall that closes one side of the bin. This test wall is supported by two vertical and three horizontal con-

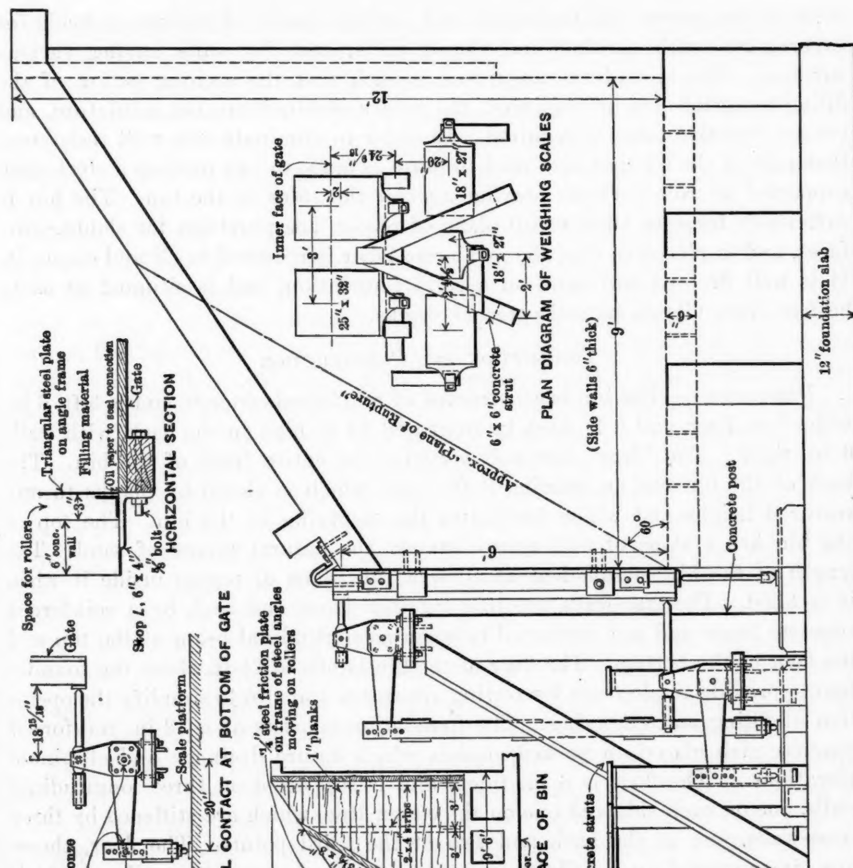

FIG. 1.—EARTH PRESSURE APPARATUS.

tacts for measuring the horizontal and vertical forces. Provision is made for experiments with vertical and sloped walls, and for walls having various surfaces. The size of the apparatus is such that the arching action of the filling material may be neglected, the values obtained are not miniature, and yet no excessive labor is required. In order to eliminate side-wall resistance, that part of the fill that may tend to move is made to rest against a steel plate supported on roller wheels bearing against the sides of the bin. The bin is sufficiently large to allow a full plane of repose, has provision for sloping surfaces, and is placed so that the minimum labor is required to fill and empty it. It is well drained and rests on a solid foundation, and is situated so as to be free from vibrations and external shocks.

Description and Construction

Dimensions.—The bin is constructed of reinforced concrete and is 5 ft. 6 in. wide, 9 ft. long, and 6 ft. high in front and 12 ft. high in the back, with walls 6 in. thick. The "free" test wall occupies the entire front of the bin. The back of the bin has an opening 3 ft. wide, which is closed by planks to any required height and which facilitates the emptying of the bin. The top of the bin has a slope of 34°, approximately the natural repose of sand. The length of the bin is sufficient to allow a full plane of repose inside it when it is filled. The side-walls are tied together front and back by a reinforced concrete beam and are supported between a longitudinal beam at the top and the floor at the bottom. The floor of the bin is placed 2½-ft. above the foundation to provide ample room for testing apparatus and also to simplify the operation of filling and emptying. The foundation consists of a 12-in. reinforced concrete slab placed on packed cinders which assure drainage and eliminate vibrations. The floor is 6 in. thick, and is supported on three longitudinal walls, one on each side and one on the center line, which are stiffened by three cross-walls, one at the back and two at the third points. The floor, therefore, is supported on a cellular construction, the forward two cells of which are open in front. All these cells are connected by drain pipes to prevent the collection of water. Drain holes were left in the floor. The entire structure above the foundation was poured monolithic. The bin was made of concrete because of the economy, a design for a steel bin having been discarded.

The "free" test wall is 6 ft. 4 in. high and 5 ft. wide, and weighs 290 lb. It is made of ⅞ in. sheathing, stiffened by four 3 by 6-in. timbers vertically and a 3 by 6-in. piece on top and 4 by 6-in. piece on the bottom, horizontally. The bin, which is 5½ ft. wide, is faced with rough timber to give a clear width of 5 ft., except in front, where the friction plates occupy the excess space.

Two bent bolts inserted through the top horizontal timber, can be hooked over a 6 by 6 by ⅜-in. angle rigidly fixed into the top of the concrete bin. In this manner, the test wall is safely held when the apparatus is not being used. Two short angles are bolted to each side stiffener, so that the face of the angles is in the plane of the inner face of the wall and the legs bear against the front of the bin walls. In this manner, the wall is prevented from moving toward the bin or against the filling material. Two short angles are fixed into the front faces of the bin walls, the faces of the outstanding legs

of the angles being flush with the inner faces of the walls. By adjusting bolts that pass through these projecting legs, with the heads toward the test wall, side or lateral motion of the gate is prevented. Such lateral movements may result from eccentricity of filling, causing unbalanced forces in the plane of the wall. By carefully watching these possible contacts, eccentricity may be detected. The method of changing the wall to take care of special cases is described in the section on tests.

Several methods for measuring the pressures were investigated, but platform scales, which are the simplest and most accurate and sensitive apparatus for measuring the forces acting on the test wall, were chosen. The deciding factor was the motion necessary to obtain readings. The platform scales used were calibrated carefully to determine the deflection of the platfrom under various loads. Two scales of 2 000 lb. capacity were used to measure the vertical components, and three scales of 1 000 lb. capacity were used for the horizontal components. The scales are equipped with a single beam having a sliding poise with set-screw. The beam, which reads to 100 lb. and has 0.5-lb. graduations, passes through a trig loop, limiting its range of motion to about 1 in., which means a movement of 0.01 in. of the platform. To decrease this movement, special stops were inserted in the scale cap over the end of the beam. These stops were shaped like an inverted T, with the horizontal arm above the beam and the vertical arm, threaded, passing through lock-nuts on the cap of the scale. By adjusting the stop, the movement of the beam can be reduced to $\frac{1}{40}$ in., the platform movement then being $\frac{1}{4\,000}$ in. Later, it was decided to double these values, because of the difficulty of detecting the point of balance when the range of motion of the beam was so small.

The Contacts.—The "free" gate is restrained by two vertical and three horizontal supports. The two vertical supports are symmetrically placed, at about the quarter-points, and rest directly on the platform scales. (See Fig. 1.) Each vertical support consists of a 1-in. steel rod, the upper end of which is pointed and bears into a steel bearing-plate screwed to the wooden wall. The lower end, which is threaded, is screwed into a steel contact point which rests on a steel plate placed on the scale platform. These contact points allow a certain amount of adjustable motion of the test wall vertically. By raising one or the other, the axis of the wall may be tipped in either direction. In this manner, the wall may be adjusted for eccentricity. When the wall was vertical, the two scales had the same initial or no load reading.

Three horizontal contacts were used to give a stable system of support. As the stress on the wall is a function of the depth, the three supports are placed in a triangle, one at the top in the center, and one in each of the lower corners. Steel plates screwed to the wall at these points formed bearing plates. Each horizontal support consisted of a strut pivoted to a bell-crank, by which arrangement the horizontal pressure was transmitted vertically to the scale platform. The bell-crank arms are in the ratio of 1 to 2, thereby decreasing the reading to one-half the actual pressure. The vertical strut consists of two 5-in. channels, at the lower end of which is an adjustable contact point, consisting of a threaded 1-in. rod in a cast shoe. All parts are made very rigid to reduce deformations to negligible values and all pivots are fitted in phosphor-

bronze bushings to reduce frictional resistance. By adjusting the threaded contact points, the test wall may be moved or tipped about its lower edge, and adjusted for any deviation of the plane of the wall from the vertical. The bell-cranks rest on steel plates fixed into stiffened reinforced concrete posts.

The scales were placed so as to make simultaneous readings possible. This required the compact arrangement shown in Figs. 1 and 2. In order to prevent any possible wobbling of the vertical supports, two lateral cross-braces in which turnbuckles were inserted, were later hooked to the rods. These turnbuckles were found to be very useful in adjusting either of the vertical contacts for deviation from the vertical.

The retarding effect of the side-walls has always introduced an element of uncertainty in earth-pressure measurements made in boxes and bins. Winkler* tried to determine the resistance of each wall by duplicating his tests with a center partition. The second set of tests was affected by four wall friction losses; whereas the first set was affected by two friction losses. This method has been shown to be inaccurate. In the apparatus, it was decided to eliminate the resistance effect of the side-walls. The first method decided on, mentioned previously, was to have these walls on rollers, permitting slight motion in a direction parallel to the wall. It was thought that this flexibility would remove the effect of forces along the sides of the fill. The objection that any such motion would cause a change in the volume and shape of the fill, as well as alter the stresses acting therein, caused the abandonment of this design. Professor Braune then suggested a method approximating actual conditions. Behind a retaining wall, a certain part of the fill tends to move toward the wall. The earth back of any given length of wall is not retarded by the earth on each side, because that too tends to move. It was decided, therefore, to have part of the side-walls of the bin movable in the direction in which the fill tends to move, that is, in a plane normal to the test wall.

The part of the fill that tends to move is approximately a triangular prism, with bases formed by the surface of the fill, the back of the wall, and the "plane of rupture". The wedge angle for sand is about 30 degrees. Two triangular steel plates of the required shape were made, each of which rests on three phosphor-bronze bushed rollers bearing against the side-walls of the bin. At the points of contact the concrete was finished very smooth. That part of the side-walls not covered by these plates is faced with timber, held to the sides by bolts through the walls, so that the bin has a uniform width of 5 ft. The steel plates are accurately counterbalanced vertically. Preliminary tests made before the plates were counterbalanced showed the necessity of eliminating the pressure due to the tendency of these plates to slip downward. Two rollers are placed at third points along the plane of rupture to eliminate the friction between the plates and the wooden facing of the walls.

In order to prevent sand flowing into the cracks, all joints were made as close as possible, thin strips of oiled paper, bent into angles, being placed along all the edges of the test wall. The joint between the steel plates and the timber was also covered with oiled paper. The area covered by this paper

* Winkler, E., in *Der Civilingenieur*, 1865, pp. 1–11.

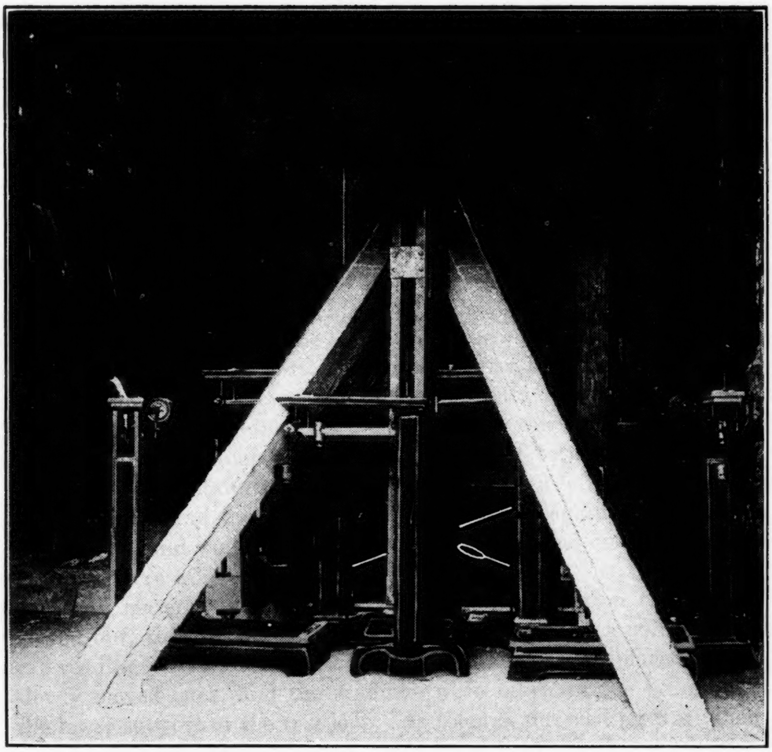

FIG. 2.—TESTING APPARATUS WITH SCALES IN PLACE.

was so small in comparison with the total exposed area that no account of its effect was taken.

The width of fill being 5 ft., losses due to the frictional resistances of the rollers back of the friction plates may be disregarded. No attempt was made to alter the shape of the plates, as the wedge angle probably averaged very nearly 30 degrees. Some of the tests were made with the plates overbalanced, that is, practically fixed. Although the resulting pressures are probably less, the decrease is not very large. The error thus incurred does not exceed the errors due to non-homogeneous fill or to slight differences or speed of filling.

A brief description of the apparatus has been published,* but because of the remarkable sensitivity and accuracy of the apparatus, this detailed description is given for the benefit of those who may plan to conduct experiments in the future. An exhaustive description of the design and method of operation is included in a thesis presented by the writer in 1921, and is in the Library of the Graduate School of the University of Cincinnati.

Method of Operation

The most important factor in the operation was to guard against obtaining passive instead of active pressures. Before each test, every contact point was loosened until the test wall was free from the bin. The axis of the wall was placed in a vertical position by adjusting the vertical supports until the two scales showed the vertical components to be equal. A plumb-bob suspended on the wall showed whether the plane of the wall was vertical. All the horizontal contacts were then tightened until the increase in readings on the scales showed that the wall was bearing against the bin. The contacts were then adjusted so as to give the minimum readings on the scales, with the wall as close to the bin as possible. In this manner, a minimum of opening between the edges of the wall and the bin was obtained. The apparatus was then left for at least 1 hour, usually over night, and the zero readings were taken again. Readings could be taken to $\frac{1}{4}$ lb., and were so taken at the beginning and end of each set—all other readings were taken to $\frac{1}{2}$ lb.

Before any material was placed in the bin, the riders on the scale-beams were moved out, that is, the scales were overbalanced, to a point more than the next reading. After a definite height of fill had been placed, the riders were slowly moved back, and the readings were taken as the beam just rose from the lower support of the trig loop. As soon as the reading was taken, the rider was run out again to a point beyond the next possible reading. This method probably reduced the motion of the gate to one-half its possible movement, or to $\frac{1}{8\,000}$ in. vertical and $\frac{1}{16\,000}$ in., horizontal, which are values less than the elastic deformation of the wall, or of any retaining wall. The pressure recorded, therefore, is the active pressure of the fill, and not the passive resistance of the fill caused by the movement of the wall.

To test the sensitiveness of the method, readings were often repeated in as short an interval of time as possible, the results being usually identical and seldom varying by $\frac{1}{2}$ lb. As the average readings were about 500 lb., and sometimes as much as 1 500 lb., this variation is negligible.

* *Engineering News-Record*, August 25, 1921.

To determine the variation of pressures caused by greater movements of the scale-beams, and, therefore, of the wall, readings of one fill were taken for the scale-beams free to move the entire height of the trig loop. In Table 1, and in all the other tables given, the scales are numbered, as follows:

No. 1.—The upper horizontal reading.
No. 2.—The lower left (facing the bin) reading.
No. 3.—The lower right (facing the bin) reading.
No. 4.—The left vertical reading.
No. 5—The right vertical reading.

The vertical readings less the zero readings give the actual vertical pressures, whereas the horizontal readings must be doubled.

TABLE 1.

	Scale Readings.				
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.
Scale-beam in restricted position	554	464	528	985	701
Scale-beam at highest point of trig loop	533	451	510	951	675
Scale-beam at middle point of trig loop	541	459	518	965	683
Scale-beam at lowest point of trig loop (same position as when restricted)	553	465	527	985	701

The total movement of the beams was $\frac{1}{2}$ in., corresponding to a movement of the wall of $\frac{1}{200}$ in. vertical and $\frac{1}{400}$ in. horizontal, which movements had no effect on the recorded pressures. Similar results were obtained in the other two tests to determine the effect of the movements of the wall. It may be concluded, therefore, that for such values, the pressures are unaffected. The effect of larger movements could not be determined, because the trig loops are not removable.

In addition to the readings of the scales, a record was kept of the temperature, time of day, state of the weather, and age of the fill in the bin. In the first few tests, readings were taken for every 3-in. increase in height of fill. This was changed to 6-in. increments, as too much time was required, both during the test and in computing results.

The five readings give the magnitude, direction, and point of application of the resultant earth pressure. The vertical component is the sum of the readings on Scales Nos. 4 and 5, less the initial readings on these two scales. The total horizontal component is twice the sum of the readings on Scales Nos. 1, 2, and 3, after the initial readings on these scales have been subtracted. The magnitude of the resultant can then be found. The direction is the angle the tangent of which is the ratio of the total vertical to the total horizontal component. By taking moments about the intersection of a plane through the vertical supports and a plane through the lower horizontal supports, the effect of these four forces is eliminated and the equality of moments caused by the upper horizontal support and by the total resultant is obtained. The upper horizontal support is 6.04 ft. above this line of intersection. Resolving the resultant into a vertical component acting along the back of the wall and a horizontal com-

ponent acting at a distance, x, above this axis of moments, and equating moments, the height, x, or the resultant pressure is obtained. A vertical wall has no moment, because the vertical supports are placed under its center of gravity. When additional parts were attached to the wall, for glass-backed walls, or oblique walls, the moment of additional parts was considered.

Tests were performed with glass and sheet metal directly fixed to the back of the wooden wall. For walls sloping toward the fill, a "false" wall was attached to the wall at three points. At the foot, it was spiked to the main wall and at the mid-point and at the top, the two walls were connected by heavy timber struts. Similar connections were used for the wall sloping away from the fill. Because of the large vertical components in the latter case, sixteen wires were also run from the lowest point of the overhanging wall to various points in the main wall to aid in transmitting the vertical component and in decreasing the deflection of the "false" wall. In both these cases, the additional parts were inside the bin, and care was taken to prevent contact on the floor or binding along the sides. The results for walls with considerable slope are not as accurate as those for vertical walls. Some of these tests were repeated to obtain check values.

A sample of the material used in each test was taken. Air-tight cans, holding about 15 lb. of material, were filled when each test was about half finished. The density, natural slope, coefficient of internal friction, coefficient of internal resistance, and moisture content of this material was later carefully determined. The coefficients of friction and resistance (kinetic and static friction) of the material of the fill on itself and on the back of the wall were also determined. The coefficient of internal resistance was measured by determining the force required to start in motion a weighed quantity of material resting on the same material. The force required to keep the mass in uniform motion gave the coefficient of internal friction. The material was placed in a cubical bottomless box that held 5½ lb. of sand, and the lower edges were beveled so that, practically, only the sand was in contact. The coefficient of internal resistance was taken as a measure of the combined effect of friction and cohesion; whereas the coefficient of internal friction was due to friction alone, as there can be no cohesion during motion. The angles corresponding to these coefficients are called the angles of internal friction and internal resistance, respectively. The angle of internal resistance in all cases (for sand) was found to be about 90% of that of natural repose.

Outline of Tests

The following is an outline of the tests performed:

I.—Variation in physical properties of the sand under various conditions.

II.—Lateral pressure of horizontal fills against a vertical wooden wall.

III.—Lateral pressure of horizontal fills against a glass-backed vertical wall.

IV.—Lateral pressure of horizontal fills against a sheet-metal-backed vertical wall.

V.—Lateral pressure of sloping fills against a vertical wooden wall.

VI.—Lateral pressure of sloping fills against a glass-backed vertical wall.

VII.—Lateral pressure of sloping fills against a sheet-metal-backed vertical wall.

VIII.—Lateral pressure of horizontal fills against walls with positive back batter.

IX.—Lateral pressure of horizontal fills against walls with negative back batter.

X.—Lateral pressure of sloping fills against a wall with a positive batter of 1:4.

XI.—Lateral pressure of sloping fills against a wall with a negative back batter of 1:6.

XII.—Lateral pressure of irregular fills against a vertical wooden wall.

XIII.—Effect of settling and changes of temperature on the lateral pressure of horizontal fills against a vertical wall.

XIV.—Effect of static loads on the lateral pressure of horizontal fills against a vertical wall.

XV.—Effect of moving loads on the lateral pressure of tamped horizontal fills against a vertical wall.

XVI.—Effect of static loads on the lateral pressure of horizontal fills against a wall with positive back batter of 1:4.

A few of these tests were made during the spring of 1921 and the remainder during the fall and winter of 1921-22. The filling and emptying of the bin eighteen times required the shoveling of about 150 tons of sand.

I.—Physical Properties of the Sand

River sand excavated from the subway cut on Canal Street, Cincinnati, Ohio, was used. When first obtained the specific gravity of the sand was about 2.6 and the moisture content was 9 per cent. Shoveled and slightly tamped into a box, it weighed 100 lb. per cu. ft. Exposure caused a slow drying, and during the last tests, the moisture content was as small as 3 per cent. The first tests, therefore, were made with a "damp" sand, the remainder with a "humid" sand in accordance with the definitions of the Special Committee to Codify Present Practice on the Bearing Value of Soils for Foundation, etc. Moisture had considerable influence on the other physical properties. A mechanical analysis is as follows:

Sieve number.................	4	8	10	14	28	48	100
Percentage passed	92	85	81	78	63	10	2½

The material contained very little clay and dirt. The natural slope was determined by pouring the sand on a table to form a cone about 12 in. high, the slope being measured along six elements. Four determinations of coefficients of friction and of resistance were made for every sample. At first, the coefficient of resistance was measured by using a spring scale and also by weights over a fixed pulley. The results were practically identical, so that only the spring-scale method was used in the later tests. A complete set of physical tests was also made on dry sand.

The unit weight of the material was a minimum (90 lb. per cu. ft.) when the moisture content was between 4 and 5 per cent. The natural slope was

greater the moister the sand, dry sand having a slope of 30° 15′ and sand with 9% moisture about 40 degrees. The same holds true for the angle of internal friction, which was 28° 15′ for dry sand and 34° for the very moist sand; and also for the angle of internal resistance, which was 28° 15′ for dry sand and 37° for the very moist sand. It should be noted that, for the dry sand, there is no difference between the angles of friction and resistance. This difference was 2° for a moisture content of 3% and 3° for a moisture content of 9 per cent. The intermediate values are fairly consistent. The difference between the internal friction and internal resistance is caused by the adhesion due to a water film around each particle of sand. This property was called "cohesion" by Coulomb, and the name is still used, although it is difficult to understand how cohesive forces can act between particles or grains of a fill separated by a film of water. This film is always present, as there are no perfectly dry materials back of retaining walls.

In order to determine whether the physical properties of the material varied with the height of the fill, two loosely built wooden boxes, with no covers, were embedded in the sand for three months, from June to September, one cube being 6 ft., and the other 10 ft., below the surface.

Physical tests on the material in these cubes and of the surface material are given in Table 2.

TABLE 2.

	Location of Material.		
	Surface.	6-ft. depth.	10-ft. depth.
Weight per cubic foot	100 lb.	98 lb.	101 lb.
Moisture content	9%	9%	9%
Angle of natural slope	40° 0	40° 40′	43° 40′
Angle of internal friction	34° 0′	34° 30′	35° 30′

Because of the small samples used, little can be concluded from these tests, and further investigation is necessary.

II.—Lateral Pressure of Horizontal Fills Against a Vertical Wooden Wall

Several tests were made during April and May, 1921, while the apparatus was being adjusted and calibrated. The following data were obtained after it was found that results could be closely duplicated and the apparatus was completed and satisfactory. In Test No. 1 (Table 3), all figures and computations are given in order to show the method. The columns contain the following:

Column 1.—Height of fill in bin.

Columns 2, 3, 4, 5, and 6.—The actual readings on Scales Nos. 1, 2, 3, 4, and 5.

Columns 7, 8, 9, 10, and 11.—The pressure recorded, obtained by subtracting the initial readings.

Column 12.—The top horizontal pressure, H_1, double the reading.

TABLE 3.—Test No. 1.

May 12, 1921, 2–5 P.M.
Weather: Rain.
Temperature: 10–12° cent. (50–55° Fahr.)
Moisture content = 9%.

Physical Properties.
Weight per cubic foot = 100 lb.
Angle of internal resistance = 37°
" " internal friction = 34°
" " natural slope = 40°
" " friction on wall (static friction) dry = 30°
" " damp = 32° 30′.

Height of fill, h, in bin, in inches.	Readings on Scales, in Pounds.					Scale Measurements, in Pounds.					Horizontal Pressures, in Pounds.			Total horizontal component, in pounds.
	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	H_1	H_2	H_3	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
0	95	60.5	60	150.5	157	....	....	....	....	....	......	......	......	......
6	96	62	64	152.5	160	1	1.5	4	2	3	2	3	8	13
9	97	66	70	159	165	2	5.5	10	8.5	8	4	11	20	35
12	97.5	78	82	178	183	2.5	17.5	22	27.5	26	5	35.	44	84
15	99	90	95.5	199.5	194	4	29.5	35.5	49	37	8	59	71	138
18	100	100	104	214	205	5	39.5	44	63.5	48	10	79	88	167
21	102	112	117	234.5	222	7	51.5	57	84	65	14	103	114	231
24	106	124.5	132.5	259	247	11	60	72.5	108.5	90	22	120	145	287
27	113	137	146	284	269	18	76.5	86	133.5	112	36	153	172	361
30	122	146.5	163	296.5	303	27	86	103	146	145	54	172	206	432
33	138	171.5	188	345	333	43	111	128	194.5	176	86	222	256	564
36	148	194	212	377	372	53	133.5	152	226.5	215	106	267	304	657
39	161	200.5	231	403	412	66	140	171	252.5	255	132	280	342	754
42	178	215	248	422.5	432	83	154.5	188	272	274	166	309	376	851
45	187	232.5	268	465	482	92	172	208	314.5	325	184	344	416	944
48	209	246	287	494.5	522	104	185.5	227	344	365	208	371	454	1 033
51	229	263	305	526.5	560.5	134	202.5	245	376	403.5	268	405	490	1 163
54	250.5	278	326.5	561	588	155.5	217.5	266.5	410.5	431	311	435	533	1 279
57	274.5	295	349.5	610.5	632	179.5	234.5	289.5	460	475	359	469	579	1 407
60	303	313	371	657	676	208	252.5	311	506.5	519	416	505	622	1 543
63	332	326.5	385	699	721	237	266	325	548.5	564	474	532	650	1 656
66	386	362	439	791	785	291	301.5	379	640.5	628	582	603	758	1 973
69	442	379	459	845.5	842.5	347	318.5	399	695	685.5	694	637	798	2 129
72	471	397	487	907	896	376	336.5	427	756.5	739	752	678	854	2 273

Column No. 1 = top horizontal contact scale reading.
Columns Nos. 2 and 3 = bottom horizontal contact scale reading.
Columns Nos. 4 and 5 = vertical contact scale reading.

TABLE 3.—(*Continued*).

Total vertical component, in pounds.	Total pressure, in pounds.	Pressures per Foot, in Pounds.			$\frac{V}{H}$ or tan ϕ'.	Direction of P, ϕ'.	Point of Application.			$x = \frac{\text{No. 25}}{H}$	$\frac{x}{h}$
		H	V	P			$6.04 H_1$.	$0.23 V$	Sum of Nos. 23 and 24.		
(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
......	....	....	....	....	.....	....	....	....	....	....	......
5	14	2.6	1.0	2.8	0.385	21° 00′	12	1	13	1.0	......
16.5	39	7.0	3.5	7.5	0.472	25° 15′	24	4	28	0.80	......
53.5	100	16.8	10.7	19.9	0.637	32° 30′	30	12	42	0.50	......
86	163	27.6	17.2	30.5	0.623	31° 55′	48	20	68	0.44	0.35
111.5	198	33.4	22.3	39.9	0.667	33° 40′	60	26	86	0.52	0.344
149	275	46.2	29.8	55.0	0.645	32° 50′	84	34	118	0.51	0.292
198.5	349	57.4	39.7	69.8	0.692	34° 40′	133	46	179	0.625	0.312
245.5	437	72.2	49.1	87.3	0.680	34° 10′	217	56	273	0.757	0.336
291	521	86.4	58.2	104.2	0.674	34° 00′	326	67	393	0.910	0.364
370.5	675	112.8	74.1	134.9	0.658	33° 20′	519	85	604	1.07	0.389
441.5	792	131.4	88.3	158.3	0.673	33° 55′	640	102	742	1.13	0.378
507.5	909	150.8	101.5	181.8	0.673	33° 55′	797	117	914	1.21	0.373
546.5	1011	170.2	109.2	202.2	0.642	32° 40′	1001	126	1127	1.33	0.380
639.5	1140	188.8	127.9	225.0	0.678	34° 10′	1111	147	1258	1.33	0.355
709	1253	206.6	141.8	250.6	0.685	34° 25′	1258	163	1421	1.38	0.345
779.5	1400	232.6	155.9	280.0	0.670	33° 50′	1620	179	1799	1.55	0.365
841.5	1531	255.8	168.3	306.2	0.659	33° 20′	1880	193	2073	1.62	0.360
935	1690	281.4	187.0	338.0	0.665	33° 40′	2152	215	2367	1.68	0.354
1025.5	1853	308.6	205.1	370.6	0.665	33° 40′	2515	236	2751	1.78	0.356
1112.5	1995	331.2	222.5	399.0	0.673	33° 55′	2862	256	3118	1.88	0.358
1268.5	2346	394.6	253.7	469.2	0.642	32° 40′	3508	292	3800	1.98	0.351
1370.5	2532	425.8	274.1	506.4	0.644	32° 50′	4190	315	4505	2.12	0.369
1495.5	2721	451.6	299.1	544.2	0.659	33° 20′	4540	344	4884	2.15	0.359

Columns 13 and *14.*—The bottom horizontal pressures, H_2 and H_3, double the readings.

Column 15.—The total horizontal component, the sum of H_1, H_2, and H_3.

Column 16.—The total vertical component, the sum of the vertical scale readings, found in Columns 10 and 11.

Column 17.—The total pressure, the square root of the sum of the squares of the two total components.

Columns 18, 19, and *20.*—The horizontal component, the vertical component, and the total pressure per foot length of wall.

Column 21.—The ratio of the total vertical to the total horizontal component.

TABLE 4.—TEST NO. 2.

May 4, 1921, 2-3 P.M., 0-3 ft.
May 5, 1921, 2-4 P.M, 3-6 ft.
Weather: Clear.
Temperature: 60-65° Fahr.

Physical Properties:
Same as in Test No. 1 (Table 3).

Height of fill, h, in inches.	TOTAL PRESSURES, IN POUNDS.				PRESSURES PER FOOT, IN POUNDS.			Direction of P, ϕ'	$\frac{x}{h}$ Ratio of height of $P:h$
	H_1.	H.	V.	P.	H.	V.	P.		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
6	20	66	4	66	13.2	0.8	13.2	3°	....
9	22	78	9	78	15.6	1.8	15.6	6°	....
12	24	100	23	103	20.0	4.6	20.6	13°	....
15	26	128	47	136	25.6	9.4	26.3	20°	....
18	30	170	80	188	34.0	16.0	37.6	25°	....
21	32	206	111	234	41.2	22.2	46.8	23° 40′	....
24	38	268	160	312	53.6	32.0	62.4	30° 50′	....
27	42	334	204	391	66.8	40.8	78.2	31° 30′	....
30	50	386	253	462	77.2	50.6	92.4	33° 10′	....
33	60	476	300	563	95.2	60.0	112.2	32° 20′	....
36	115	650	387	757	130.0	77.3	151.4	29° 55′	....
39	154	762	439	880	152.4	87.8	176.0	30° 00′	....
42	166	798	486	934	159.6	97.2	186.8	31° 20′	....
45	190	878	555	1 040	175.6	111.0	208.0	32° 20′	0.398
48	244	1 026	654	1 215	205.0	130.8	242.3	32° 50′	0.404
51	318	1 222	768	1 443	244.4	153.6	288.6	32° 10′	0.395
54	352	1 302	823	1 541	260.4	164.6	308.2	32° 10′	0.389
57	396	1 400	893	1 661	280.0	176.6	332.2	32° 30′	0.391
60	469	1 562	992	1 851	312.4	198.3	370.1	32° 20′	0.391
63	556	1 706	1 071	2 016	341.2	214.2	403.2	32° 10′	0.402
66	700	1 986	1 224	2 333	397.2	244.8	466.6	31° 40′	0.404
69	752	2 112	1 344	2 502	422.4	268.8	500.4	32° 30′	0.400
72	834	2 292	1 443	2 708	458.4	288.6	541.6	32° 10′	0.390

Column 22.—The angle the tangent of which is the ratio of the total vertical to the total horizontal component. Columns 23 to 27 furnish data on the point of application of the total pressure, by the formula developed for each type of test wall. The results are compared with three theories. The Coulomb theory, in which the wall friction is disregarded, is shown by the coefficient of $(\frac{1}{2}\, y\, h^2)$ as N. In the case of a vertical wall, it is equal to the horizontal component and takes no account of a vertical component. In the cases of inclined walls, the horizontal component, as given by this theory, is denoted by N_H. The general wedge theory, in which the friction on the wall is taken into account, is shown by the coefficient of $(\frac{1}{2}\, y\, h^2)$ as C. The horizontal component is H. The Rankine theory is shown by the coefficient of

($\frac{1}{2} y h^2$) as R. The horizontal component is K. It was decided to compare the theories by the coefficients of the quantities ($\frac{1}{2} y h^2$), in order to compare the results with hydrostatic pressures, y being the density of, and h the height of, the fill.

To find the height of the total pressure, note that the intersection of the plane through the vertical contacts and through the lower horizontal contacts is level with the bottom of the fill. The upper horizontal contact is 6.04 ft. above this axis; the back of the wall (along which the vertical component, V, acts), is 0.23 ft. from the axis. Calling x the height of the horizontal component,

$$H x = 6.04\ H_1 + 0.23\ V,$$

or

$$x = \frac{6.04\ H_1}{H} + \frac{0.23\ V}{H}.$$

The weight of the wall has no effect, because the vertical supports are below the center of gravity. The large number of data in each test makes it impractical to present all the tests in full. Except for the first test, only the essential data are presented.

The bin was filled in two days. The readings for the height of 36 in. are the averages of the readings at the end of the first day and at the beginning of the second. The high values for $\frac{x}{h}$ are due to the settling of the fill over night (see Test XIII for the effect of settling). The low values of ϕ' at the beginning of the test are caused by the dry wall.

TABLE 5.—Showing Comparison Between Experiment and Theories for the Case of a Horizontal Fill and a Vertical Wall.

(See Fig. 3)

Height of fill in bin, in inches.	Pressures per Foot of Wall, in Pounds.						$\frac{1}{2} y h^2$ $y = 100$ h = feet	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
	Test No. 1.		Test No. 2.		Average.				
	H.	*P.*	*H.*	*P.*	*H.*	*P.*			
12	16.8	19.9	20.0	20.6	18.4	20.3	50	0.368	0.406
15	27.6	30.5	25.6	26.3	26.6	28.4	78	0.341	0.369
18	33.4	39.9	34.0	37.6	33.7	38.8	112.5	0.300	0.345
21	46.2	55.0	41.2	46.8	43.6	50.9	153	0.284	0.332
24	57.4	69.8	53.6	62.4	55.5	66.1	200	0.278	0.331
27	72.2	87.3	66.8	78.2	69.5	82.9	253	0.275	0.328
30	86.4	104.2	77.2	92.4	81.7	98.3	312	0.262	0.315
33	112.8	134.9	95.2	112.2	104.0	123.6	378	0.276	0.327
36	131.4	158.3	130.0	151.4	130.7	154.9	450	0.290	0.344
39	150.8	181.8	152.4	176.0	151.6	178.9	529	0.287	0.338
42	170.2	202.2	159.6	186.8	164.9	194.5	611	0.270	0.318
45	188.8	225.0	175.6	208.0	182.2	216.5	702	0.260	0.309
48	206.6	250.6	205.2	242.3	205.9	246.5	800	0.257	0.308
51	232.6	280.0	244.4	288.6	238.5	284.3	902	0.265	0.316
54	255.8	306.2	260.4	308.2	253.1	307.2	1 012	0.250	0.301
57	281.4	338.0	280.0	332.2	280.7	335.1	1 128	0.249	0.297
60	308.6	370.6	312.4	370.1	310.5	370.3	1 250	0.248	0.296
63	331.2	399.0	341.2	403.2	336.2	401.1	1 380	0.244	0.291
66	394.6	469.2	397.2	466.6	395.9	468.9	1 510	0.262	0.310
69	425.8	506.4	422.4	500.4	424.1	503.4	1 650	0.257	0.305
72	454.6	544.2	458.4	541.6	456.5	542.9	1 800	0.254	0.302
Averages from 24 in.								0.264	0.314

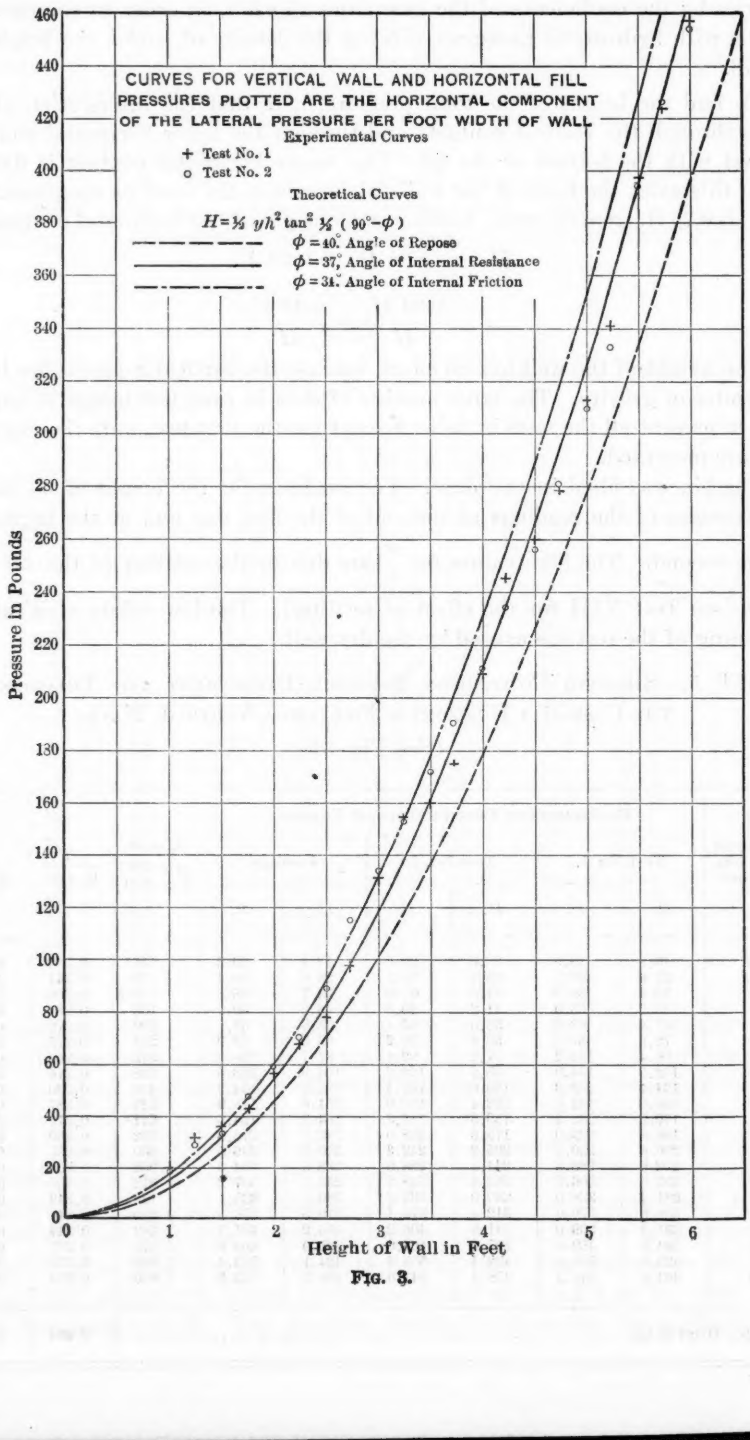
CURVES FOR VERTICAL WALL AND HORIZONTAL FILL
PRESSURES PLOTTED ARE THE HORIZONTAL COMPONENT
OF THE LATERAL PRESSURE PER FOOT WIDTH OF WALL
Experimental Curves
+ Test No. 1
o Test No. 2
Theoretical Curves
H = ½ γh² tan² ½ (90° − φ)
φ = 40° Angle of Repose
φ = 37° Angle of Internal Resistance
φ = 34° Angle of Internal Friction
Pressure in Pounds
Height of Wall in Feet
0
1
2
3
4
5
6
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
420
440
460

FIG. 3.

The Coulomb and Rankine formulas are:

$$P = \tfrac{1}{2}\, y\, h^2 \tan^2 \tfrac{1}{2}\,(90° - \phi) = N\,(\tfrac{1}{2}\, y\, h^2) = R\,(\tfrac{1}{2}\, y\, h^2).$$

Both theories give a horizontal resultant.

If we take

$\phi = 40°$, the natural slope, $R = N = \tan^2 25° \quad = 0.218$

$\phi = 37°$, the internal resistance, $N = \tan^2 26° \tfrac{1}{2} = 0.248$

$\phi = 34°$, the internal friction, $\quad N = \tan^2 28° \quad = 0.283$

The total pressure in the general wedge theory for this case is:

$$\frac{N}{\cos \phi'} = C_1 = \frac{0.248}{\cos 32°\ 30'} = 0.294;$$

for ϕ = internal resistance; ϕ' = friction along the wall. In C_1, the nature of the wall has been assumed to have no effect on the horizontal component. Assuming that it has, for $\phi = 37°$, $\phi' = 32°\ 30'$, $n = 0.818$, $C_2 = 0.230$.

$$H = C_2 \cos \phi' = 0.193; \quad n = \sqrt{\frac{\sin(\phi + \phi')\sin\phi}{\cos\phi'}}.$$

Conclusions.—Tests Nos. 1 and 2 give the same results, and agree quite closely. It is evident that a vertical component exists. The average of all the readings gives 33° for the angle of inclination of the resultant pressure. The fill was quite moist, and the wooden wall, when it was exposed after the test, was quite damp. The coefficient of friction between the sand and a wet yellow pine board, across the grain (identical with the case of the test wall), was found to be 32° 30′.

The point of application varies from $0.350\,h$ to $0.400\,h$, depending on conditions. The cause and nature of this variation, as well as that of the inclination, are considered in subsequent tests.

For this case, the Rankine and Coulomb theories give the same formula, which seems to give the amount of the horizontal component very closely, if the value given to ϕ is the angle of internal resistance. Low values are obtained by using ϕ as the angle of natural slope. Using ϕ as the angle of internal friction gives high values, which is to be expected, as the effect of cohesion can then be disregarded. The average value for $\dfrac{H}{\frac{1}{2}\, y\, h} = 0.264$, which corresponds almost exactly to $N = \tan^2 \tfrac{1}{2}\,(90° - \phi)$, in which $\phi = 36$ degrees. The experimental value of the angle of internal resistance was 37 degrees. The general wedge theory gives low values both for the horizontal component and for the total pressure, but the ratio is the same as was found. The curves shown on Fig. 3 give a graphical comparison between the two experimental results, the Coulomb-Rankine theory for ϕ = natural slope; and the formula,

$$H = \tfrac{1}{2}\, y\, h^2 \tan^2 \tfrac{1}{2}\,(90 - \phi_1),$$

in which ϕ_1 is the angle of internal resistance. It was first assumed that ϕ should be the angle of internal friction. Graphical comparison with the experimental curves showed a constant difference between theory and experiment, evidently due to disregarding the effect of cohesion. Introducing the effect of the cohesion into the value of ϕ has solved the difficulty.

III.—Lateral Pressure of Horizontal Fills Against a Glass-Backed Vertical Wall

Closely fitting sheets of glass were fastened to the back of the wooden wall to a height of 24 in. above a 2-in. strip of wood nailed at the bottom of the wall, and this held the glass in place and prevented slipping. Accuracy could not be expected because of the small heights used, and the method of fastening.

TABLE 6.—Test No. 3.

October 8, 1921.
Weather = Clear.
Temperature = 12° cent. (54° Fahr.)
Moisture Content = 4.9%

Physical Properties:
Weight per cubic foot = 93 lb.
Angle of Natural Slope = 35°30′
" " Internal Resistance = 33°50′
" " Internal Friction = 31°50
" " Wall Friction = 28°10′

h in inches.	Total Pressures, in Pounds.				Pressures, in Pounds per Foot.			Tan $\phi' = \frac{V}{H}$	ϕ'	$\frac{H}{\frac{1}{2} y h^2}$
	H_1.	H.	V.	P.	H.	V.	P.			
12	2.5	61.5	46.5	77	12	9	15	0.755	37°	0.528
15	5.5	107.5	74.5	153	22	15	31	0.694	35°	0.304
18	9.5	144.5	98.5	175	29	20	35	0.681	34°	0.276
21	14	172	122.5	211	34	25	42	0.712	35°	0.240
24	23	263.5	172.5	314	53	35	63	0.655	33°	0.285
Average										0.273

$$C_1 = \tan^2 \tfrac{1}{2}(90 - \phi_1) = \tan^2 28°05' = 0.284.$$
$$C = \tan^2 \tfrac{1}{2}(90 - \phi) = \tan^2 27°15' = 0.265.$$

In general, the conclusions from the test with a wooden wall hold also for a smoother wall. The large value of the angle of inclination is due to the method of holding the glass in place; the bottom clamp allowing the fill to rest directly on the wall. It should be noted that ϕ' decreases as the height of the fill increases, and a greater proportion of glass surface is effective. This is well shown in the test of sloped surfaces against a glass wall.

IV.—Lateral Pressure of Horizontal Fills Against a Sheet-Metal-Backed Vertical Wall

In order to determine whether the nature of the wall had any effect on the lateral pressure, sheet metal with a clean, polished surface was nailed to a height of 3 ft. directly to the wall.

The results check the conclusions given previously. The lateral horizontal pressure is, therefore, independent of the nature of the wall. The high value for the angle of inclination was expected. The surface of the sheet metal was rusted and roughened by the contact and rubbing of the damp sand.

V.—Lateral Pressure of Sloping Fills Against a Vertical Wooden Wall.

Two tests were run to determine the lateral pressure of non-horizontal fills against a vertical wall. In the first Test, No. 5, the bin was filled to a depth

of 5 ft., with the surface sloping at the maximum possible angle below the horizontal. The fill was then changed, in intervals of about 3°, until the greatest possible slope above the horizontal was obtained. In this manner, a comparison is had of all possible surfaces with a fixed height of wall. In the second test, No. 6, the surface slope was kept constant and different heights of fill were tested. It was found to be impractical to use the angle of repose as the slope to be tested, because the material would slip and the larger particles would roll down to the wall. A slope as steep as possible was used. Further tests with sloped surfaces and broken surfaces are described subsequently. (See Fig. 4.)

TABLE 7.—Test No. 4.

October 9, 1921.
Weather : Clear.
Temperature = 17° cent. (63° Fahr.)
Moisture in Fill = 4.2%

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38° 30′
" Internal Resistance = 37° 30′
" Internal Friction = 34° 50′
" Wall Friction = 28° 00′

h, in inches.	H_1.	Total Pressures, in Pounds.			Pressures, in Pounds per Foot.			Inclination. ϕ'	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{V}{\frac{1}{2} y h^2}$
		H.	*V.*	*P.*	*H.*	*V.*	*P.*			
14	4	81	51.5	96	16	10	19	32° 25′	0.250	0.297
17	5.5	106.5	74.5	128	21	15	26	35° 00′	0.224	0.277
20	8.5	160.5	97.5	188	32	20	38	31° 15′	0.244	0.290
23	13	220	135	258	44	27	52	31° 35′	0.255	0.301
26	20.5	279.5	168	326	56	34	65	31° 00′	0.254	0.295
29	31	354	210.5	412	71	42	82	30° 45′	0.258	0.298
32	41.5	415.5	245.5	483	83	49	95	30° 30′	0.248	0.284
35	58	501	293.5	580	100	59	116	30° 20′	0.251	0.291
38	78	584	340	675	117	68	135	30° 15′	0.248	0.286
Averages..........								31° 00′	0.248	0.291

$N = R = \tan^2 \frac{1}{2} (90 - \phi) = \tan^2 25° 45' = 0.232.$

$N_1 = R_1 = \tan^2 \frac{1}{2} (90 - \phi_1) = \tan^2 26° 15' = 0.243.$

$C = \frac{N_1}{\cos \phi'} = \frac{0.243}{0.883} = 0.275.$
(taking $\phi' = 28°$)

$C = \frac{0.243}{0.857} = 0.284 \; (\phi' = 31°).$

Table 8, which shows the results of Test No. 5, furnishes the following data: The value of $h = 5$ ft; α, the inclination of the wall is zero. In Column 1 is given values of the surface inclination, ϵ, negative below the horizontal and positive above it. As it was difficult to obtain plane surfaces without considerable disturbance of the fill, a rod was fixed at the proper height across the bin, 8 ft. back of the wall, and the sand filled to a plane surface by eye. The surfaces below the horizontal were probably somewhat concave, whereas those above the horizontal were convex.

Column 2 gives the top horizontal pressure. The first two readings were probably a little too high, because of the impact of the fill as it was thrown against the wall.

Columns 3, 4, and 5 give the total horizontal and vertical components and the total pressure on the wall.

Column 6 gives the angle of inclination of the resultant.

Columns 7, 8, and 9 give the horizontal and vertical components and the total pressure per foot of width of the wall.

Column 10 gives the ratio of the height of inclination to the total height of fill. This value is found in the same way as in Tests Nos. 1 and 2.

Column 11 gives the coefficient of ($\frac{1}{2}\, y\, h^2$) for the experimental horizontal component per foot of wall.

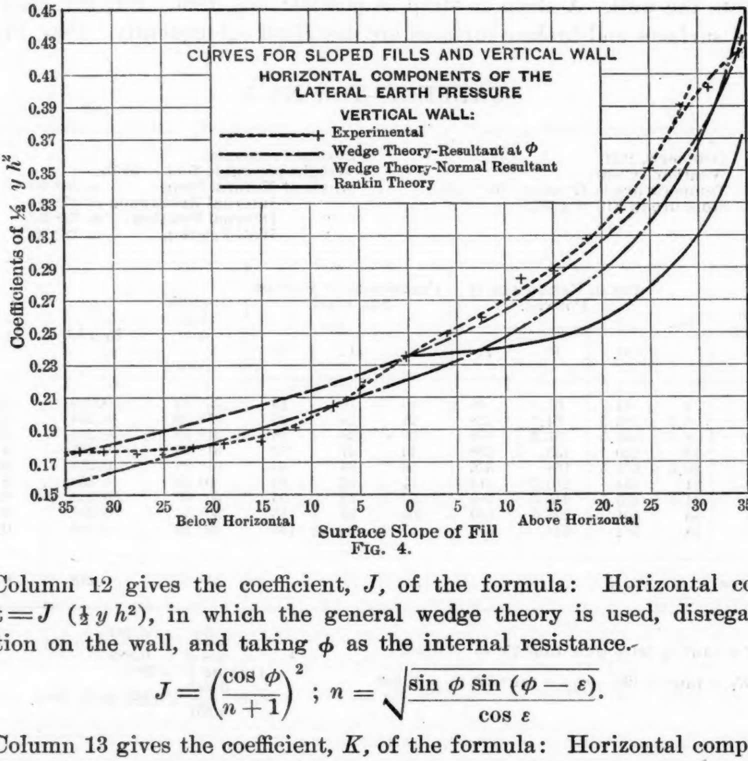

FIG. 4.

Column 12 gives the coefficient, J, of the formula: Horizontal component $= J\ (\frac{1}{2}\, y\, h^2)$, in which the general wedge theory is used, disregarding friction on the wall, and taking ϕ as the internal resistance.

$$J = \left(\frac{\cos\phi}{n+1}\right)^2\ ;\ n = \sqrt{\frac{\sin\phi\,\sin(\phi-\varepsilon)}{\cos\varepsilon}}.$$

Column 13 gives the coefficient, K, of the formula: Horizontal component $= K\ (\frac{1}{2}\, y\, h^2)$, in which the Rankine theory is used. The theory does not apply for negative slopes. ϕ is taken as the angle of internal resistance.

$$K = \cos^2\varepsilon\,\frac{\cos\varepsilon - \sqrt{\cos^2\varepsilon - \cos^2\phi}}{\cos\varepsilon + \sqrt{\cos^2\varepsilon - \cos^2\phi}}$$

By comparing the horizontal components (in this case, also the normal components), the different assumptions as to the direction of the total pressure are eliminated.

Column 14 gives the coefficient, H', of the formula: Horizontal component $= H'\ (\frac{1}{2}\, y\, h^2)$, in which the general wedge theory is used, taking into account the friction of the wall. ϕ is taken as the angle of internal resistance.

$$H' = \left(\frac{\cos\phi}{n+1}\right)^2\ ;\ n = \sqrt{\frac{\sin(\phi+\phi')\,\sin(\phi-\varepsilon)}{\cos\phi'\,\cos\varepsilon}}.$$

TABLE 8.—TEST NO. 5.

(See Fig. 4)

September 25, 1921.
September 27, 1921.
Weather: Cloudy and rain.
Temperature: 22° cent. (72° Fahr.)
h = 5 ft., a = 0.

Physical Properties:
Weight per cubic foot = 95 lb.
Angle of Natural Slope = 39° 30′
" " Internal Resistance = 38°
" " Internal Friction = 35° 30′
" " Wall Friction = 32° 30′

Surface slope, ϵ.	Total Pressure, in Pounds.				Inclination of P. ϕ'	Pressure per Foot, in Pounds.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2}y\,h^2}$	J.	K.	H'.	Percent, area wedge.	Percent. H (*Exp.*)
	H_1.	H.	V.	P.		H.	V.	P.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
−33° 40′	255	1064	679	1265	32° 30′	213	136	253	0.33	0.179	0.179	.....	0.163	76	78
−31°	253	1058	678	1260	32° 30′	212	136	252	0.33	0.179	0.183	.....	0.163	77	78
−28° 05′	251	1052	679	1252	33°	210	136	250	0.33	0.177	0.187	.....	0.171	79	77
−25°	251	1052	685	1255	33°	210	137	251	0.33	0.177	0.191	.....	0.176	81	77
−21° 50′	255	1063	689	1270	33°	213	138	254	0.33	0.179	0.196	.....	0.181	84	78
−18° 30′	257	1089	719	1305	33°	218	144	261	0.33	0.184	0.201	.....	0.186	86	80
−14° 55′	258	1101	730	1320	33° 30′	220	146	264	0.33	0.185	0.208	.....	0.192	89	80
−11° 20′	287	1144	778	1385	34°	229	156	277	0.33	0.193	0.213	.....	0.198	93	84
− 7° 40′	299	1229	810	1470	33° 30′	246	162	284	0.33	0.204	0.217	.....	0.205	94	90
− 3° 50′	331	1314	868	1570	33°	263	174	314	0.34	0.221	0.225	.....	0.212	97	96
0	365	1412	914	1680	32° 30′	287	183	336	0.34	0.237	0.237	0.237	0.220	100	100
+ 3° 50′	399	1487	946	1760	33°	297	189	352	0.35	0.250	0.253	0.239	0.228	102	108
+ 7° 40′	416	1547	1011	1850	32° 30′	309	202	370	0.35	0.260	0.264	0.241	0.239	107	113
+11° 20′	517	1679	1080	1945	32° 30′	336	216	389	0.40	0.282	0.273	0.244	0.250	111	123
+14° 55′	532	1712	1100	1990	32° 30′	342	220	398	0.40	0.288	0.285	0.249	0.264	115	125
+18° 30′	569	1833	1170	2160	32° 30′	367	234	432	0.40	0.308	0.296	0.253	0.279	119	134
+21° 50′	608	1939	1240	2305	32° 30′	388	248	461	0.42	0.327	0.314	0.266	0.295	124	142
+25°	680	2079	1330	2480	32° 30′	416	266	496	0.40	0.351	0.332	0.278	0.314	129	152
+28° 05′	796	2319	1500	2610	33°	464	300	550	0.44	0.391	0.352	0.296	0.344	135	169
+31°	804	2344	1530	2650	33°	469	306	556	0.44	0.395	0.391	0.319	0.377	142	203
+34°	834	2504	1600	2900	32° 30′	501	320	594	0.43	0.423	0.425	0.362	0.432	149	216
Averages.										0.257	0.253	.....	0.242	.....	.....

Column 15 gives the percentage ratio of the area of the theoretical wedge of rupture to the area of the wedge when the surface is horizontal. The plane of rupture was assumed to bisect the angle between the vertical and a line drawn at the angle of internal resistance to the horizontal.

$$A = \tfrac{1}{2}\, h^2 \frac{\sin \varpi \cos \varepsilon}{\sin (90^\circ - \varepsilon - \omega)},\quad \varpi = \text{the angle of the wedge.}$$

In this case, $\phi = 38^\circ$, $\omega = 26^\circ$.

Column 16 gives the percentage ratio of the horizontal component to the horizontal component when the fill is level. The same filling material was used in Test No. 6, and the fill was placed at an angle of 34° above the horizontal, up to a depth of 3 ft.

TABLE 9.—TEST NO. 6.

Date: September 22, 1921.
Weather: Clear.
Temperature: 21° cent. (70° Fahr.)

Physical Properties,
Same as for Test No. 5.
Fill at $\epsilon = 34^\circ$.

h, in inches.	TOTAL PRESSURES, IN POUNDS.				PRESSURES, IN POUNDS PER FOOT.			Inclination of $P = \phi'$	$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
	H_1.	H.	V.	P.	H.	V.	P.				
12	13	112	43	120	24	9	24	....	....	0.53	0.53
18	15.5	163	103	192	33	21	38	32°	0.48	0.33	0.36
24	25	373	211	429	75	42	86	30°	0.28	0.419	0.480
30	40.5	501	300	584	100	60	117	31°	0.33	0.356	0.416
36	102	756	473	891	151	95	178	32°	0.33	0.373	0.440
Averages 2 ft. to 3 ft.										0.382	0.444

The theoretical coefficients of $(\frac{1}{2}\, y\, h^2)$ are:

$H = 0.334$; $C = 0.396$ for the wedge theory, wall friction being considered.

$N = 0.384$ for the wedge theory, wall friction being disregarded.

$K = 0.333$, $R = 0.402$ for the Rankine theory.

The coefficient for the total pressure, corresponding to $N = 0.384$, and $\phi' = 32^\circ\, 30'$, as in Test No. 5, is 0.455.

For $\phi' = 31^\circ$, as seems to be case in this test, the coefficient is 0.445.

Conclusions.—For a vertical wall, with any inclination of the surface of the fill, the value of the lateral earth pressure is given in:

(*a*) Magnitude, by the formulas derived by the wedge theory for the horizontal component, assuming the pressure to be normal to the wall, that is, disregarding the wall friction. This might be expected, as the previous tests showed that the nature of the wall surface had no effect on the horizontal component. The vertical component is equal to the horizontal component multiplied by the coefficient of friction on the wall.

(*b*) Direction, by the angle of friction between the fill and the surface of the wall.

(*c*) Point of application, by the value, 0.33 h to 0.40 h, depending on conditions. These conditions are discussed in later tests.

The point of application of the resultant for fills below the horizontal is at $\frac{1}{3}h$, and is higher for fills above the horizontal, reaching $0.40\,h$ for fills having the greatest slopes. This test proves the presence of a vertical component for all cases. Rankine's theory would give an upward component for all the cases of negative ϵ. Columns 15 and 16 were computed in order to compare the variation of pressure with that of the wedge area. The pressures seem to vary much more than the area of the wedge, especially at the greater slopes. The value of the pressure, therefore, is a function of a number of variables, including the area of the wedge, and not on the area of the wedge itself, as is assumed by some. This is based on the assumption of a plane of rupture which bisects the angle between the vertical and the line drawn at the angle of internal resistance to the horizontal. The general wedge theory furnishes low values, because the friction of the wall is assumed to diminish the pressure.

VI.—Lateral Pressures of Sloping Fills Against a Vertical Glass-Backed Wall.

A few readings were taken of the pressure of a fill, 2 ft. high and inclined at 31° to the horizontal, against a glass-backed vertical wall.

The pressure was 107 lb. per ft. of width, the horizontal component was 92 lb., or a coefficient of 0.500. Theory gives $N = 0.495$, for $\phi_1 = 33°\ 50'$, $\epsilon = 31°$. The angle of inclination was about 30°, which is somewhat more than the coefficient of friction on the glass wall, due to the obstructions caused by clamps on the wall. The results agree with the conclusions given in Section V.

VII.—Lateral Pressures of Sloping Fills Against a Sheet-Metal-Backed Vertical Wall.

In order to determine whether the conclusions of Section V were affected by changing the wall, the test for fills above the horizontal was repeated with a sheet-metal wall, the height of the fill being 3 ft.

TABLE 10.—Test No. 7.

Date October 9, 1921.
Weather: Clear.
Temperature = 17° cent. (63° Fahr.)
Height of fill = 3 ft.
Wall Vertical.

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38°
" Internal Resistance = 34° 30′
" Internal Friction = 32°
" Wall Friction = 28°

Surface slope, ϵ	Total Pressures, in Pounds.				Pressures, in Pounds per Foot.			Inclination of P ϕ'	$\frac{x}{h}$	$\frac{H}{\frac{1}{2}y\,h^2}$
	H_1.	H.	V.	P.	H.	V.	P.			
0	122	585	340	675	117	68	135	30°	0.333	0.280
+ 4° 45′	127	601	350	695	120	70	139	30°	0.339	0.297
+11° 20′	143	669	397	779	134	79	156	30° 30′	0.348	0.316
+16° 40′	167	764	434	884	153	87	177	29° 30′	0.358	0.337
+21° 50′	206	895	495	1 045	175	99	209	29°	0.375	0.366
+26° 30′	237	996	544	1 138	199	109	228	28° 30′	0.402	0.409
+27° 40′	270	1105	592	1 252	221	118	250	28°	0.415	0.423
Averages								28° 45′	......	0.347

The same conclusions may be drawn from the results given in Table 10 as from those of Test No. 6. The resultant acts at the angle of friction on the wall, and above $\frac{1}{3}h$, especially for the greater slopes. The amount of the horizontal component is given by the wedge theory, which disregards the effect of the friction of the wall. Sections V, VI, and VII show that the horizontal component of the pressures of any kind of fill on a vertical wall is independent of the nature of the wall surface.

Table 11 is presented in order that the coefficient of $(\frac{1}{2} y h^2)$ as found experimentally may be compared with those values found by the various theories, for horizontal components.

TABLE 11.

Experiment:	0.280	0.297	0.316	0.337	0.366	0.409	0.423
N (wedge theory)	0.277	0.288	0.311	0.336	0.363	0.403	0.416
K (Rankine theory)	0.277	0.277	0.284	0.296	0.317	0.349	0.362
H (wall friction theory)	0.220	0.228	0.254	0.279	0.308	0.347	0.363
ϵ (surface slope)	0	4°45′	11°20′	16°40′	21°50′	26°30′	27°40′

VIII.—Lateral Pressures of Horizontal Fills Against Wooden Walls with Positive Back Batter.

In this test, four walls were used, with backs sloping at 1 : 12 (4° 45′), 1 : 8 (7° 15′), 1 : 6 (9° 30′), and 1 : 4 (14° 00′). The fixed parts of the recording apparatus required the entire test wall to be placed inside the bin. The desired shape of wall was constructed and nailed to the inside face of the wooden vertical wall. Several of the tests were repeated because sand, leaking through cracks and getting under the wall, decreased the vertical component. In the repeated tests, it was found that this trouble had little effect on the horizontal readings, as the friction under the wall was quite small.

A preliminary test was made, with a 1 : 4 wall, to determine the direction of the resultant. Special care was taken in this test to have the wall free. The fill was about 4 ft. high, but because of several changes in its shape, made to prevent leakage, no attempt to deduce the magnitude of the pressure from the data of this test is made. The height of fill was always measured vertically.

TABLE 12.—Actual Pressures, in Pounds.

	Top horizontal.	Bottom horizontal.	Vertical.	Total horizontal.	$\frac{V}{H}$	$(\phi' + \alpha)$
a	87	708	714	795	0.895	41°55′
b	85	714	739	799	0.925	42°45′
c	85	712	716	797	0.898	41°55′
d	81	716	749	797	0.940	43°10′
e	83	714	737	797	0.925	42°45′
f	83	714	742	797	0.930	42°00′
g	84	715	727	799	0.910	42°20′
Average					0.918	42°25′

The space beneath the wall was cleaned after each reading. This procedure necessitated raising the wall about $\frac{1}{100}$ in. each time, and lowering it again; however, the readings are quite consistent.

The resultant is inclined to the horizontal at an angle of 42° 25′ and the wall slopes 14° 00′ from the vertical, so that the resultant makes an angle of 28° 25′ to the normal to the wall. The angle of wall friction, as determined experimentally for this sand (93 lb. per cu. ft.; moisture content 3.2%; $\phi_1 =$ 31°), was slightly less than 29 degrees. This proves that on a wall inclined with positive batter the resultant acts at an angle to normal, which angle is equal to the angle of friction between the wall and the fill.

To determine the magnitude and point of application of the resultant, a test was made on each wall with horizontal fills, and also with oblique fills (described subsequently) on the 1 : 4 wall.

The readings and the computation of the components and of the total pressure were the same as in the tests with the vertical walls. In addition, the values of the normal and tangential components (N and T) on the wall are given. The relations between the P (total pressure), H (horizontal component), V (vertical component), N, T, ϕ' (angle of wall friction), and α (angle of inclination of the wall from the vertical), are:

$$P = \sqrt{H^2 + V^2}; \qquad \tan(\phi' + \alpha) = \frac{V}{H}$$

$$H = P\cos(\phi' + \alpha); \quad N = P\cos\phi' = \frac{H\cos\phi'}{\cos(\phi' + \alpha)}$$

$$V = P\sin(\phi' + \alpha); \quad T = P\sin\phi' = \frac{H\sin\phi'}{\cos(\phi' + \alpha)}$$

In computing the height of the resultant, the moment of the wall about the vertical supports had to be considered, as the wall was no longer balanced.

The height of the resultant,

$$x = \frac{6.04\ H_1 + V\left(0.48 + \dfrac{6}{a}\right) + V_0\left(0.48 + \dfrac{2}{a}\right)}{H + \dfrac{V}{a}}$$

For, taking moments about the vertical supports,

$$H\,x = H_1\,(6.04) + 0.48\ V + V(6 - x)\,\frac{1}{a} + V_0\left(0.48 + \frac{6}{3\,a}\right);$$

in which

H = total horizontal component;
H_1 = top horizontal component with lever arm of 6.04;
V = total vertical component, taken to act at the same point on the back of the wall as H;
V_0 = additional weight of the wall due to false wall;
a = batter ratio (1 : a).

$$V \text{ acts at a distance } \left(0.48 + \frac{6 - x}{a}\right) \text{ ft. from the axis.}$$

$$V_0 \text{ acts at a distance } \left(0.48 + \frac{6}{3\,a}\right) \text{ ft. from the axis.}$$

assuming the additional piece to be a solid triangular wedge of base $\frac{6}{a}$ ft.

TABLE 13.—Test No. 8.

Date: November 17, 1921.
Weather: Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = + 1 : 12 (4°45′)
Moisture Content = 3.5%

Physical Properties:
Weight per cubic foot = 96 lb.
Angle of Natural Slope = 32°45′
" " Internal Resistance = 29°40′
" " Internal Friction = 27°30′
" " Wall Friction = 29°.

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P $\frac{x}{h}$	Coefficients of ($\frac{1}{2} y h^2$)		
	H.	*V.*	*P.*	*N.*	*T.*		*H.*	*P.*	*N.*
2.5	70	47	84	73	44	0.39	0.234	0.280	0.244
3.	92	62	112	93	54	0.39	0.213	0.260	0.215
3.5	125	84	151	132	74	0.405	0.213	0.257	0.225
4.	154	104	187	163	90	0.396	0.203	0.247	0.215
Averages						0.395	0.216	0.271	0.225

$V = H \tan(\phi' + \alpha)$; $T = N \tan(\phi')$.

TABLE 14.—Test No. 9.

Date: November 20, 1921.
Weather: Clear.
Temperature = 9.5° cent. (49° Fahr.)
Wall Slope = + 1 : 8 (7° 15′)
Moisture Content = 3.5%

Physical Properties:
Weight per cubic foot = 96 lb.
Angle of Natural Slope = 32°45′
" " Internal Resistance = 29°40′
" " Internal Friction = 27°30′
" " Wall Friction = 29°

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P $\frac{x}{h}$	Coefficients of ($\frac{1}{2} y h^2$).		
	H.	*V.*	*P.*	*N.*	*T.*		*H.*	*P.*	*N.*
2.5	69	49	85	74	41	0.345	0.230	0.283	0.247
3.	88	64	108	95	53	0.337	0.204	0.250	0.220
3.5	115	84	142	124	69	0.378	0.195	0.241	0.211
4.	148	108	184	160	89	0.420	0.193	0.239	0.208
Averages						0.370	0.208	0.253	0.222

In order to compare the experimental results with the various theories, the following coefficients were computed for each type of wall. The Rankine theory, as originally given, does not apply to a sloped wall. The general wedge theory, assuming the resultant to act at an angle, ϕ', to the normal to the wall, that is $(\phi' + \alpha)$ to the horizontal, gives,

$$P = \tfrac{1}{2}\,y\,h^2\,(C) \text{ and } H = \tfrac{1}{2}\,y\,h^2\,(H)$$

for the total and horizontal components. For a level fill,

$$(C) = \left(\frac{\cos(\phi - \alpha)}{(n + 1)\cos\alpha}\right)^2 \frac{1}{\cos(\phi' + \alpha)};\ n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi)}{\cos(\alpha)\cos(\phi' + \alpha)}}.$$

$$(H) = C\cos(\phi' + \alpha).$$

The wedge theory, which assumes the resultant pressure to act normal to the wall, gives $N = \frac{1}{2}yh^2(N)$ as the normal component, in which,

$$(N) = \left(\frac{\cos(\phi - \alpha)}{(n + 1)\cos\alpha}\right)^2 \frac{1}{\cos\alpha} = \left(\frac{\cos(\phi - \alpha)}{\sin\phi + \cos\alpha}\right)^2 \frac{1}{\cos\alpha},\ n = \frac{\sin\phi}{\cos\alpha}.$$

TABLE 15.—Test No. 10.

Date: November 21, 1921.
Weather: Clear.
Temperature = 10° cent. (50° Fahr.)
Wall Slope = + 1:6 (9°30'
Moisture Content = 3.7%

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Natural Slope = 36°
" " Internal Resistance = 32°
" " Internal Friction = 29°40'
" " Wall Friction = 29°

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P.	Coefficients of ($\frac{1}{2}yh^2$).		
	H.	V.	P.	N.	T.	$\frac{x}{h}$	H.	P.	N.
2.5	61	48	78	73	34	0.388	0.202	0.254	0.241
3	81	65	104	91	50	0.373	0.186	0.238	0.208
3.5	121	97	155	136	75	0.392	0.204	0.261	0.229
4	157	125	194	176	97	0.405	0.203	0.250	0.227
Averages..........						0.390	0.199	0.251	0.226

TABLE 16.—Test No. 11.

Date: November 23, 1921.
Weather: Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = + 1:4 (14°00')
Moisture Content = 3.7%

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Natural Slope = 35°
" " Internal Resistance = 31°
" " Internal Friction = 29°15'
" " Wall Friction = 29°

Height of fill, in feet.	Pressures, per Foot of Wall, in Pounds.					Height of P.	Coefficients of ($\frac{1}{2}yh^2$).		
	H.	V.	P.	N.	T.	$\frac{x}{h}$	H.	P.	N.
2.5	93	86	127	111	61	0.390	0.308	0.420	0.367
3	125	117	172	150	83	0.388	0.287	0.394	0.344
3.5	152	142	209	182	101	0.409	0.256	0.352	0.306
Averages..........						0.396	0.284	0.389	0.339

The ratio between the experimental and theoretical values of H equals 0.790; of C equals 0.795; and of N equals 0.65; therefore the tests seem to show that the general wedge theory, taking into account the wall friction, applies to battered walls and a horizontal fill within 20 per cent. The resultant is inclined to the normal at an angle equal to that of wall friction. The resultant acts at about $\frac{3}{8}h$. The error in these tests seems quite large, unless all theories are disregarded as giving too high results. The writer does not believe that the error of these tests is greater than 10% in any one case.

TABLE 17.—COMPARISON OF EXPERIMENT WITH THEORY—INCLINED WALLS.
(See Fig. 5, Horizontal Fill, Heel of Wall in Fill.)

Slope of wall.	EXPERIMENTAL.						THEORETICAL.				EXPERIMENT.	
	a	ϕ'	y.	H.	C.	N.	C.	H.	N.	J.	$\frac{x}{h}$	ϕ
+1:12	4° 45′	29°	96	0.216	0.271	0.225	0.338	0.281	0.371	0.370	0.395	29° 40′
+1: 8	7° 15′	29°	96	0.208	0.253	0.222	0.360	0.290	0.383	0.380	0.370	29° 40′
+1: 6	9° 30′	29°	97	0.199	0.251	0.226	0.255	0.278	0.374	0.368	0.390	32° 00′
+1: 4	14° 00′	29°	97	0.284	0.389	0.339	0.413	0.302	0.428	0.415	0.396	31° 00′
Averages....................				0.227	0.291	0.253	0.367	0.288	0.389	0.383	0.388	......

IX.—Lateral Pressure of Horizontal Fills against a Wooden Wall with Negative Back Batter.

In this test, five walls were used, with backs sloping at 1:12 (4°45′), 1:6 (9°30′), 1:4 (14°00′); 1:3 (18°25′); and 1:2 (27°15′). The wall was constructed in the same manner as that described in Section VIII. As the lower end of the wall was an edge, the upper end overhanging, there was no trouble in attaching the extra wall to the original vertical wall, and no possibility of sand leaking under the wall. These tests are, therefore, more accurate.

The readings and the computation of the pressures are the same as for the vertical walls. In computing the height of the resultant, the moment of the wall about the vertical supports had to be taken into account, because the wall was not absolutely balanced. The height of the resultant, x, is given by the formula:

$$x = \frac{6.04\ H_1 + 0.48\ V + V_0\ \left(0.48 + \dfrac{4}{3\ a}\right)}{H - \dfrac{V}{a}};$$

The derivation of this formula is the same as that given in Section VIII.

TABLE 18.—TEST No. 12.

Date : October 30, 1921.
Weather : Clear.
Temperature : 18° cent. (65° Fahr.)
Wall Slope = —1:12, (4° 45′); ($\phi' + a$) = 25°.
Moisture Content = 3.65%

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Natural Slope = 34° 50′
" Internal Resistance = 30° 30′
" Internal Friction = 27° 55′
" Wall Friction = 29° 45′

Height of fill, h, in feet.	H_1.	TOTAL PRESSURES, IN POUNDS.			($\phi' + a$)	PRESSURES PER FOOT.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
		H.	V.	P.		H.	V.	P.			
2	45	204	84	220	22°	41	17	44	0.370	0.216	0.232
2.5	67	298	127	308	23°	60	25	62	0.344	0.201	0.208
3	105	458	207	503	24°	92	41	101	0.343	0.214	0.235
3.5	152	639	307	710	26°	128	61	142	0.340	0.218	0.246
4	219	892	441	995	26° 20′	178	88	199	0.377	0.234	0.262
4.5	238	1067	532	1190	26° 30′	213	106	238	0.375	0.222	0.248
Averages....................................					24° 38′	....			0.358	0.221	0.239

TABLE 19.—Test No. 13.

Date : November 6, 1921.
Weather : Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = —1 : 6, (9° 30′); $(\phi' + \alpha)$ = 19° 30′.
Moisture Content = 3.25%

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38° 20′
" Internal Resistance = 31° 50′
" Internal Friction = 28° 30′
" Wall Friction = 29° +

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds. H.	V.	P.	$(\phi' + \alpha)$	Pressures per Foot. H.	V.	P.	$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
2.5	26	306	91	320	16° 30′	61	18	64	0.40	0.208	0.218
3	43	424	139	445	18° 10′	85	28	89	0.35	0.201	0.210
3.5	74	565	199	599	19° 20′	113	40	120	0.34	0.196	0.208
4	126	645	256	695	21° 40′	129	51	139	0.40	0.172	0.185
Averages				....	19° 00′	....	....	....	0.37	0.195	0.205

TABLE 20.—Test No. 14.

Date : November 10, 1921.
Weather : Clear.
Temperature = 7.5° cent. (47° Fahr.)
Wall Slope = —1 : 4, (14°) ; $(\phi' + \alpha)$ = 13°.
Moisture Content = 3.5%.

Physical Properties :
Weight per cubic foot = 97 lb.
Angle of Internal Resistance = 31°.
" Natural Slope = 34°.
" Internal Friction = 29°.
" Wall Friction = 27°.

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds. H.	V.	P.	$(\phi' + \alpha)$.	Pressures per Foot. H.	V.	P	$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
2.5	20	260	58	266	12° 25′	52	12	53	0.40	0.173	0.177
3	39	372	84	380	12° 35′	74	17	76	0.32	0.167	0.171
3.5	88	545	123	560	12° 45′	109	25	112	0.36	0.185	0.190
4	134	695	154	710	12° 30′	139	31	142	0.36	0.183	0.187
Averages					12° 34′	......	......	......	0.36	0.177	0.181

TABLE 21.—Test No. 15.

Date : November 11, 1921.
Weather : Cloudy and Rain.
Temperature = 8° cent. (46° Fahr.)
Wall Slope = —1 : 3, (18° 25′) ; $(\phi' + \alpha)$ = 11° 35′.
Moisture Content = 3.55%

Physical Properties :
Weight per cubic foot = 97 lb.
Angle of Internal Resistance = 30° 20′
" Natural Slope = 36° 20′
" Internal Friction = 27° 30′
" Wall Friction = 30°

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds. H.	V.	P.	$(\phi' + \alpha)$	Pressures per Foot. H.	V.	P.	$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
2.5	18	269	67	276	14°	54	13	55	0.40	0.181	0.185
3	38	365	80	373	12° 20′	73	16	75	0.39	0.171	0.175
3.5	79	488	101	499	11° 40′	98	20	100	0.39	0.167	0.170
4	122	619	128	630	11° 10′	124	26	126	0.40	0.168	0.166
Averages					12° 53′	......	......	......	0.395	0.171	0.174

TABLE 22.—Test No. 16.

Date: November 13, 1921.
Weather: Cloudy.
Temperature = 6.5° cent. (44° Fahr.)
Wall Slope = −24.75 : 48, (27° 15′); ($\phi' + \alpha$) = 2° 45′.
Moisture Content = 3.55%.

Physical Properties:
Weight per cubic foot = 97 lb.
Angle of Internal Resistance = 30° 20′
" Natural Slope = 36° 30′
" Internal Friction = 27° 30′
" Wall Friction = 30°

Height of fill, h, in feet.	H_1.	Total Pressures, in Pounds.			($\phi' + \alpha$)	Pressures per Foot.			$\frac{x}{h}$	$\frac{H}{\frac{1}{2} y h^2}$	$\frac{P}{\frac{1}{2} y h^2}$
		H.	V.	P.		H.	V.	P.			
2.5	11	184	14	185	4° 30′	37	3	37	0.40	0.124	0.124
3	16	235	21	236	5°	47	4	47	0.35	0.110	0.110
3.5	30	300	29	301	5° 30′	60	6	60	0.40	0.102	0.102
4	55	378	41	381	8° 20′	76	8	76	0.40	0.100	0.100
Averages					5° 50′	......	......	......	0.39	0.109	0.109

It was noticed that the sand seemed to "creep" away from the wall. There seemed to be a space, between the sand and the wall, at least 1 ft. deep, and about ½ in. wide. As the depth of fill increased, the lower part probably had a better grip on the wall, hence the increase in the value of ($\phi' + \alpha$).

In order to compare the experimental results with the various theories, the coefficients were computed as in Section VIII. The formulas are the same, but the value of α is now negative. Instead of the normal pressure, N, the horizontal component, J, of the normal pressure is given: $J = N \cos \phi'$.

TABLE 23.—Comparison of Experiment with Theory. Inclined Walls. (See Fig. 5.) Horizontal Fill. Top of Wall Overhanging.

Slope of wall.	ϕ	ϕ'	y	Direction of P.		Experiment.		Theory.			Experiment.
				Experiment.	Theory.	H.	P.	H.	P.	J.	$\frac{x}{h}$
1 : 12	30° 36′	29° 45′	97	24° 38′	25°	0.221	0.239	0.229	0.256	0.295	0.358
1 : 6	31° 50′	29°	94	19° 00′	19° 30′	0.195	0.205	0.197	0.210	0.245	0.37
1 : 4	31° 00′	29°	97	12° 34′	13°	0.177	0.181	0.189	0.194	0.226	0.36
1 : 3	30° 20′	30°	97	12° 53′	11° 35′	0.171	0.174	0.172	0.176	0.208	0.395
1 : 2	30° 20′	30°	97	5° 50′	2° 45′	0.109	0.109	0.113	0.113	0.144	0.39
Averages				14° 59′	14° 22′	0.175	0.182	0.180	0.190	0.224	0.375

The total pressure is very closely given by the general wedge formula, taking into consideration the wall friction. Its direction is always at an angle, ϕ', to the normal, and its height at about ⅜ h. The height of application seems to rise with an increase in the inclination of the wall.

X.—Lateral Pressure of Sloping Fills against a Wall with a Positive Back Batter of 1 : 4.

As in the tests with a vertical wall, various surfaces, with a constant height of wall of 3 ft., were tested. For the method of computation see Section VIII. In calculating the theoretical values, the following formulas were used:

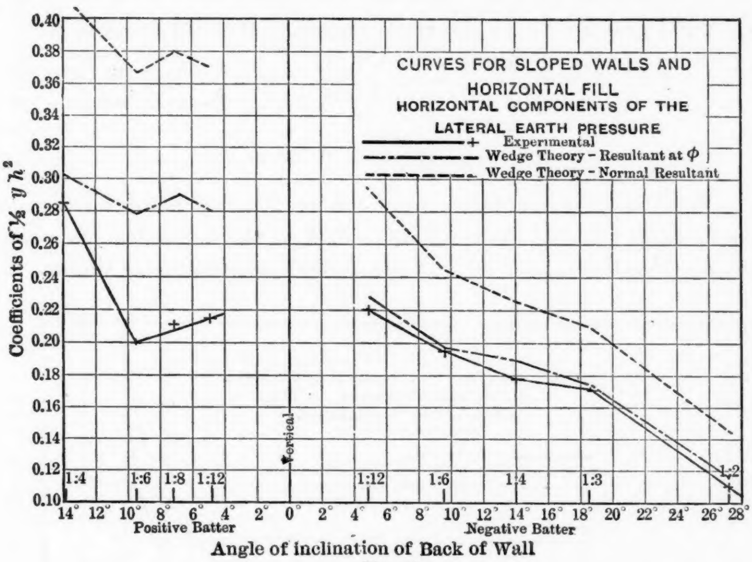

FIG. 5.

The general wedge theory, taking into consideration the wall friction, is:

$$E = \tfrac{1}{2}\, y\, h^2 C, \quad C = \left(\frac{\cos(\phi - \alpha)}{(n+1)\cos\alpha}\right)^2 \frac{1}{\cos(\phi' + \alpha)},$$

$$n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi - \varepsilon)}{\cos(\phi' + \alpha)\cos(\alpha - \varepsilon)}}$$

The horizontal component coefficient, $H = C \cos(\phi' + \alpha)$.

TABLE 24.—TEST NO. 17.
(See Fig. 6.)

Date: November 24, 1921.
Weather: Clear.
Temperature = 14° cent. (57° Fahr.)
Wall Slope = 1 : 4, (14°).
Moisture Content = 3.7%.

Physical Properties:
Weight per cubic foot = 93 lb.
Angle of Natural Slope = 35°
" Internal Resistance = 31°
" Internal Friction = 29° 15′
" Wall Friction = 29°

$(\phi' + \alpha)$ 43°. $\tfrac{1}{2}\, y\, h^2 = 419$

Surface slope. ϵ	Pressures, in Pounds per Foot.					Height of P. $\frac{x}{h}$	Coefficients of $\frac{1}{2} y h^2$.					
							Experimental.			Theoretical.		
	H.	*V.*	*P.*	*N.*	*T.*		*H.*	*P.*	*N.*	*H.*	*N.*	*J.*
−41° 10′	63	58	86	75	42	0.33	0.151	0.206	0.179	0.168	0.266	0.258
−32° 00′	77	72	106	93	50	0.33	0.184	0.253	0.222	0.195	0.299	0.290
−18° 25′	90	84	124	108	60	0.34	0.215	0.296	0.258	0.235	0.351	0.340
−12° 30′	100	94	139	120	66	0.34	0.239	0.332	0.287	0.255	0.373	0.362
− 6° 20′	110	102	150	131	73	0.35	0.263	0.358	0.313	0.276	0.400	0.388
Level	124	115	170	148	82	0.376	0.296	0.405	0.354	0.302	0.428	0.415
6° 20′	132	123	180	157	87	0.396	0.315	0.430	0.375	0.336	0.464	0.450
12° 30′	140	131	192	168	93	0.415	0.335	0.459	0.401	0.380	0.514	0.494
18° 25′	155	145	213	186	103	0.416	0.370	0.509	0.445	0.425	0.557	0.540
20° 30′	160	149	219	191	106	0.420	0.382	0.523	0.456	0.451	0.583	0.565
26° 30′	166	155	228	199	110	0.439	0.397	0.545	0.474	0.571	0.685	0.665
30° 15′	178	166	244	213	118	0.446	0.425	0.583	0.510	0.765	0.863	0.837
Averages							0.298	0.408	0.357	0.363	0.482	......

The normal pressure wedge theory, disregarding the wall friction, is:

$$E = \tfrac{1}{2}\, y\, h^2 N, \quad N = \left(\frac{\cos(\phi - \alpha)}{(n + 1)\cos\alpha}\right)^2 \frac{1}{\cos\alpha} \cdot n = \sqrt{\frac{\sin\phi \sin(\phi - \varepsilon)}{\cos\alpha \cos(\alpha - \varepsilon)}}$$

The direction of P is $\cos^{-1}\dfrac{H}{P} = \dfrac{298}{408} = 0.732$ or $43°$.

The point of application varies from $\frac{1}{3}\,h$ to $\frac{4}{9}\,h$, depending on the slope. The experimental value of the resultant is 82% of the theoretical, as given by the general wedge theory, and 74% of the normal pressure wedge theory.

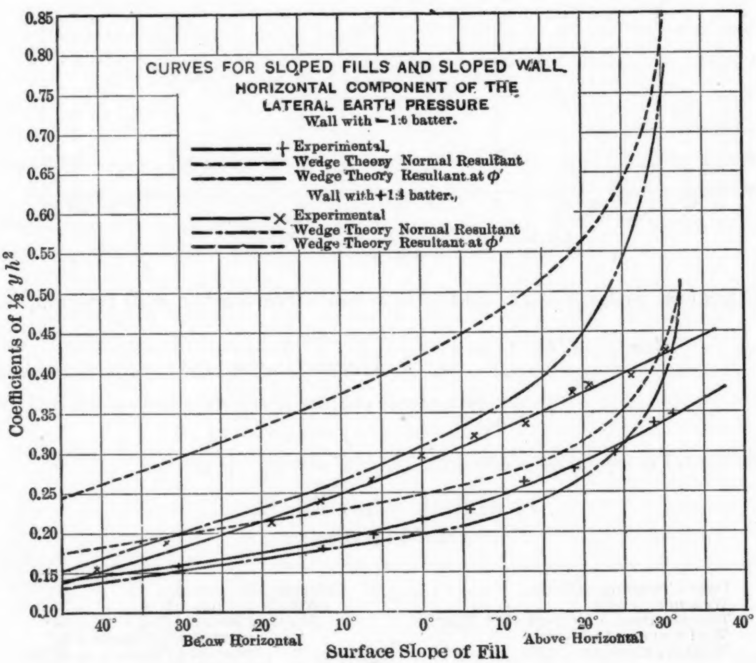

FIG. 6.

XI.—Lateral Pressure of Sloping Fills against a Wall with a Negative Back Batter of 1 : 6.

A test similar to the previous one was made with a wall having an overhanging top, that is, a negative batter. The computations and theoretical formulas are the same as in Sections IX and X.

The direction of the resultant to the horizontal is $(\phi' + \alpha)$, or at an angle ϕ', to the normal. The point of application varies, but is always above the third point and rises with increased slopes of fill. The average magnitude of the resultant is given very accurately by the general wedge theory, the theoretical values being about 5% lower than the experimental values, except for the very steep slopes above the horizontal, in which case the experimental are much lower than the theoretical values.

TABLE 25.—TEST No. 18.
(See Fig. 6.)

Date: November 8, 1921.
Weather = Clear.
Temperature = 12° cent. (54° Fahr.)
Wall slope = —1 : 6, (9° 30′).
Moisture Content = 3.3%

Physical Properties:
Weight per cubic foot = 94 lb.
Angle of Natural Slope = 38° 20′
" " Internal Resistance = 32°
" " Internal Friction = 28° 20′
" " Wall Friction = 30°

$h = 3$ ft. $(\phi' + a) = 20° 30°$, $\frac{1}{2}y\,h^2 = 423$.

Surface slope, ϵ		Pressures per Foot, in Pounds. H.	V.	P.	Inclination of P. $(\phi' + a)$	$\frac{x}{h}$	$\frac{H.}{\frac{1}{2}y\,h^2}$	$\frac{P.}{\frac{1}{2}y\,h^2}$	Theoretical Coefficients. H.	C.	J.
—38° 20′	19 : 24	63	23	67	20° 00′	0.398	0.149	0.158	0.139	0.148	0.187
—30° 15′	1 : 2	67	24	72	19° 40′	0.382	0.158	0.170	0.153	0.164	0.198
—12° 30′	2 : 9	76	26	80	19° 10′	0.353	0.180	0.189	0.176	0.187	0.223
— 6° 20′	1 : 9	82	31	87	20° 50′	0.358	0.194	0.205	0.186	0.199	0.233
Level	....	91	33	97	19° 55′	0.358	0.215	0.229	0.198	0.211	0.245
6° 20′	1 : 9	98	36	104	20° 10′	0.385	0.231	0.246	0.211	0.225	0.260
12° 30′	2 : 9	109	40	117	20° 20′	0.365	0.258	0.277	0.232	0.248	0.280
18° 25′	1 : 3	117	44	125	20° 40′	0.374	0.276	0.296	0.258	0.276	0.305
24° 00′	4 : 9	128	48	136	20° 40′	0.385	0.300	0.321	0.297	0.317	0.343
29° 00′	5 : 9	141	54	151	21° 00′	0.368	0.333	0.357	0.369	0.394	0.407
31° 30′	11 : 18	147	57	158	21° 10′	0.370	0.347	0.374	0.475	0.507	0.495
Averages.					20° 20′	....	0.240	0.257	0.245	0.261	0.289

XII.—Lateral Pressure of Irregular Fills against a Vertical Wooden Wall.

This test was made to determine the presence or absence of a wedge of rupture. The bin was filled to a level of 6 ft. and several readings were taken. Starting at the back, 9 ft. from the wall, the fill was raised 1 ft. Readings were taken as the increased height of fill approached the wall. In this manner, the rate of increase in lateral pressure was determined. If there is a fixed prism of rupture, the increase in fill beyond the plane of rupture will have no effect on the lateral pressure, but an increase in fill between the wall and the plane of rupture will cause an increase in pressure. The increase in fill was governed by placing a board across the bin at the required distance from the back, 1 ft., 2 ft., etc., successively. Sand was shoveled behind this board to a height of 1 ft., and readings were taken. This procedure practically corresponded to a uniform load of 100 lb. per sq. ft. moving toward the wall. After these readings were taken, the board was removed, which allowed the sand to take a position of natural repose. The distance from the wall to the toe of the slope was about 6 in. less than the distance from the wall to the previous vertical face of the increased fill. Readings were taken after this change.

The report of the test in Table 26 gives the horizontal and vertical components and the resultant pressure per foot of width of wall for each increment of fill in both shapes, together with the percentage increases in each value. Assuming the plane of repose to be 36° to the horizontal, and the plane of rupture to bisect the angle between the vertical wall and the plane of repose, the areas of the wedges between the wall and the two inclined planes were computed. These values include the additional fill. The percentage increases in these are also given.

Conclusion.—The test shows the presence of a "wedge" or "prism" of rupture, closely approximating the theoretical wedge. Loading beyond the theoretical plane of rupture had practically no effect on the pressure. As soon as the load crossed the plane of rupture, the horizontal and vertical components increased, the vertical somewhat faster than the horizontal. The variation in the total pressure is very close to that in the theoretical wedge of rupture. The height of the resultant remained practically unchanged. It is evident that all the material above the plane of repose does not act as the wedge of rupture. Whether or not the surface of rupture is a plane is difficult to determine. There was no indication of a break in the top surface of the fill in any of the tests. Such a break could hardly be expected from the small movements of the wall.

TABLE 26.—TEST No. 19.

Date: May 19, 1921
Weather = Clear.
Temperature = 28° cent. (82° Fahr.)

Physical Properties = same as in Test No. 1.
Original Fill: 6 ft., level.

Additional fill 1 ft. high.	Distance from Wall, in Feet.	Pressures, per Foot, in Pounds.			Height of *P*.		Percentage Increases.						
		H.	*V.*	*P.*	$\frac{V.}{H.}$	$\frac{x.}{h.}$	*H.*	*V.*	*P.*	*r.**	*u.*†	$\frac{x.}{h.}$	$\frac{V.}{H.}$
None	....	467	296	552	0.633	0.368	0.0	0	0	0	0.0	0	0.0
Vert	8	468	296	554	0.633	0.367	0.2	0	0	3.5	0	0	0
Sloped	7.5	468	297	555	0.633	0.367	0.2	0	1	3.5	0	0	0
Vert	7	468	297	555	0.633	0.367	0.2	0	1	7.0	0	0	0
Sloped	6.5	468	297	555	0.633	0.367	0.2	0	1	7.0	0	0	0
Vert	6	470	297	556	0.633	0.368	1.0	0	1	11.0	0	0	0
Sloped	5.5	470	298	556	0.633	0.368	1.0	1	1	11.0	0	0	0
Vert	5	471	299	558	0.634	0.369	1.0	1	1	15	0	0	0
Sloped	4.5	471	299	558	0.634	0.369	1.0	1	1	15	0	0	0
Vert	4	473	301	561	0.635	0.369	1	2	2	19	0	0	0
Sloped	3.5	473	301	561	0.636	0.370	1	2	2	19	0	0	0
Vert	3	481	316	576	0.655	0.369	3	7	4	23	3.5	0	3.5
Sloped	2.5	484	320	580	0.662	0.368	4	8	5	23	3.5	0	4.5
Vert	2	499	341	605	0.684	0.361	7	15	9	27	14.5	—2	8
Sloped	1.5	500	341	605	0.682	0.360	7	15	9	27	14.5	—2	8
Sloped	0	569	374	680	0.656	0.380	22	26	23	36	28	+3	4
Sums							50	78	61	250	64	——	28

* Area between wall and plane of repose for 6-ft. level: 24.78 sq. ft.
† Area between wall and plane of rupture for 6-ft. level: 9.18 sq. feet.

XIII.—Effect of Settling and Changes of Temperature on the Lateral Pressure of Horizontal Fills against a Vertical Wall.

Several tests were conducted to determine the effect of changes in temperature, humidity, etc., and to investigate the effect of settling. Readings were taken about 1 hour apart. One test consisted in taking readings for a period of 8 days, during which there was a temperature variation of 13° cent. (23° Fahr.), and several rains.

A "blank test", in which readings on the scales were caused by a mechanically applied pressure against the walls, showed very little variation, thereby proving that the variations in the following tests are due to changes in the fill and not to changes in the apparatus. In these tests, only those readings are given in which changes occurred. During the 8-day test mentioned previously,

80 sets of readings, distributed in such a manner that the effect of extremes of temperature and humidity could be investigated, were taken.

The maximum (x) and minimum (n) values are given in Table 27.

TABLE 27.

Age of fill, in hours.	Time of day.	*H.*	*V.*	*P.*	$\frac{V}{H}$	$\frac{x}{h}$
0	5 P.M.	456	229 (x)	545	0.658	0.355 (n)
14	12 M.	419 (n)	286	508 (n)	0.680 (x)	0.363
88-90	9 A.M.	465	278 (n)	542	0.597	0.370
90	11 A.M.	468	278	544	0.594 (n)	0.371
114	11 A.M.	473 (x)	286	553 (x)	0.603	0.374 (x)
Percentage variations:						
Of maximum............		3.7	0	1.5	3.3	5.4
Of minimum............		8.1	6.3	6.8	9.7	0
Total variations...		11.8	6.3	8.3	13.0	5.4

TABLE 28.—Test No. 20, Time Test.

Date: May 12-19, 1921.
Wooden wall—vertical.
Horizontal fill 6 ft. high.

Physical Properties, same as in Test No. 1.

Day.	Hour.	Age of fill, in hours.	Temperature, in degrees, cent.	Pressures, per Foot, in Pounds.			Tan ϕ' $= \frac{V}{H}$	$\frac{x}{h}$	Percentage Changes.				Weather and Humidity.
				H.	*V.*	*P.*			*H.*	*V.*	*P.*	$\frac{x}{h}$	
1st.....	5 P.M.	0	12	456	299	545	0.658	0 355	......	......	......	......	Rain.
	12 M.	19	14	419	286	508	0.680	0.363	−8.1	−3.9	−6.8	+2.3	Storm
2d.....	2 P.M.	21	12	422	286	509	0.678	0.364	−7.4	−3.9	−6.6	+2.5	Clear,
	6 P.M.	25	12	422	286	509	0.679	0.361	−7.4	−3.9	−6.6	+1.7	"
	9 A.M.	40	19	426	280	510	0.660	0.363	−6.6	−5.7	−6.4	+2.3	"
3d.....	12 M.	43	20	432	284	517	0.658	0.365	−5.3	−4.5	−5.1	+2.8	"
	4 P.M.	47	20	432	286	518	0.662	0.364	−5.3	−3.9	−5.0	+2.5	"
	11 A.M.	66	16	450	279	530	0.620	0.364	−1.3	−6.0	−2.8	+2.5	"
4th....	12 M.	67	17	451	279	530	0.620	0.365	−1.1	−6.0	−2.8	+2.8	"
	12:30 P.M.	67.5	17	453	279	532	0.616	0.365	−0.7	−6.0	−2.4	+2.8	"
	8 A.M.	87	13	460	279	538	0.607	0.366	+0.9	−6.0	−1.3	+3.1	"
	9 A.M.	88	15	465	278	542	0.597	0.370	+2.0	−6.3	−0.6	+4.2	"
	10 A.M.	89	15	466	278	543	0.599	0.371	+2.2	−6.3	−0.4	+4.5	"
	11 A.M.	90	17	468	278	544	0.594	0.371	+2.6	−6.3	−0.2	+4.5	"
	12 M.	91	17	469	280	545	0.595	0.372	+2.8	−5.7	0.0	+4.8	"
5th....	1 P.M.	92	17.5	469	280	547	0.596	0.372	+2.8	−5.7	+0.4	+4.8	"
	2 P.M.	93	17.5	469	282	547	0.599	0.371	+2.8	−5.1	+0.4	+4.5	"
	3 P.M.	94	17.5	469	282	547	0.601	0.370	+2.8	−5.1	+0.4	+4.2	"
	5 P.M.	96	17	467	283	546	0.606	0.370	+2.4	−4.8	+0.2	+4.2	"
	6 P.M.	97	16.5	466	283	545	0.608	0.370	+2.2	−4.8	0.0	+4.2	"
	10 P.M.	101	14	469	283	548	0.604	0.369	+2.8	−4.8	+0.6	+4.0	"
	8 A.M.	111	14	461	281	540	0.608	0.369	+1.1	−5.4	−0.9	+4.0	"
	9 A.M.	112	16	463	280	541	0.605	0.371	+1.5	−5.7	−0.7	+4.5	"
	10 A.M.	113	17	466	282	545	0.605	0.373	+2.2	−5.1	0.0	+5.1	"
6th....	11 A.M.	114	19	473	286	553	0.603	0.374	+3.7	−3.9	+1.5	+5.4	"
	12 M.	115	19	471	288	552	0.612	0.374	+3.3	−3.3	+1.3	+5.4	"
	2 P.M.	117	19	469	290	551	0.617	0.373	+2.8	−2.7	+1.1	+5.1	"
	5 P.M.	120	18	465	292	549	0.628	0.372	+2.0	−2.1	+0.7	+4.8	"
	8 A.M.	135	17	450	287	534	0.638	0.368	−1.3	−3.6	−2.0	+3.1	"
7th....	12 M.	139	20.5	452	289	536	0.640	0.372	−0.9	−3.0	−1.7	+4.8	"
	8 P.M.	147	20	452	294	539	0.650	0.366	−0.9	−1.5	−1.1	+3.1	"
	8 A.M.	159	19	443	292	530	0.659	0.363	−2.8	−2.1	−2.8	+2.3	"
8th....	11 A.M.	162	23	446	291	533	0.652	0.366	−2.2	−2.4	−2.2	+3.1	"
	12 M.	163	23.5	451	291	534	0.650	0.366	−1.8	−2.4	−2.0	+3.1	"
	2 P.M.	165	26	451	294	539	0.651	0.364	−1.1	−1.5	−1.1	+2.5	Warm.

Conclusions from this test may be stated as follows:

1.—The pressure just after filling is practically the maximum.

2.—A rise in temperature is accompanied by an increase in both components, and an increase in the height of the resultant.

3.—A drop in temperature is accompanied by a decrease in both components and a decrease in the height of the resultant.

4.—The ratio of vertical to horizontal components seems to be unaffected by the age of the fill or by changes in temperature. It is, however, affected by the humidity, because of changes in the water content.

5.—The height of the resultant varies as just noted, and also slowly rises as the age of the fill increases. For a fill with horizontal top surface, the height of the resultant seems to be a maximum at $\frac{3}{8}\,h$.

6.—Settling of fill is accompanied by a slight decrease in pressure, with intermittent sudden increases, due to small ruptures in the fill. These increases, however, soon vanish.

7.—The minimum pressures occur soon after the filling. (Tests described subsequently show this more clearly than those given in this section).

8.—Curves plotted for this test give an average variation of the pressure of 0.15% per degree centigrade, taking the average for each day.

The curves of "pressure with time" are similar to the curve of "damped vibrations". The total variation in the horizontal component of 11.8% is worthy of note. A short report of the preliminary tests on the effect of surcharges has been published.* It may be of interest to note that this article, as well as the preliminary description of the apparatus have both appeared in German technical periodicals.†

TABLE 29.—TEST NO. 21, TIME TEST.

Date: May 19-23, 1921
Weather—Clear.
Time Test on a Tamped Fill; 7 ft. High; 6 ft. vertical Wall. The fill was 168 hours old when the test began.

Physical Properties: same as Test 1.

Day.	Hour.	Age of Fill, in hours.	Temperature, in degrees, centigrade.	Pressures per Foot, in Pounds.			$\tan\phi' = \frac{V}{H}$.	$\frac{x}{h}$	Percentage Changes in:			
				H.	*V.*	P.			*H.*	*V.*	*P.*	$\frac{x}{h}$
1st...	4 P.M.	0	26	569	374	680	0.659	0.380	....	....	....	....
	5 P.M.	1	25	570	378	684	0.664	0.381	+0.2	+1.1	+0.6	+0.3
2d....	8 A.M.	16	21	560	373	682	0.665	0.383	−1.6	−0.3	+0.3	+0.8
	12 M.	20	24	566	373	678	0.659	0.385	−0.5	−0.3	−0.3	+1.3
	1 P.M.	21	25	565	373	677	0.660	0.385	−0.7	−0.3	−0.4	+1.3
	2 P.M.	22	26	561	375	682	0.658	0.379	−1.4	+0.3	+0.3	−0.3
	5 P.M.	25	25	583	376	690	0.645	0.379	+2.5	+0.5	+1.5	−0.3
3d....	10 A.M.	42	25	568	371	678	0.654	0.378	−0.2	−0.8	−0.3	−0.5
	1 P.M.	45	25	569	370	678	0.651	0.377	0	−1.1	−0.3	−0.8
5th...	1 P.M.	93	27	563	366	671	0.651	0.377	−1.1	−2.1	−1.3	−0.8

The variation in tamped fills seems to be very much less than in ordinary fills. The temperature range in this test was only 5°C. (41° Fahr.), so that large variations cannot be expected.

* *Engineering News-Record*, January 19, 1922.

† *Der Bauingenieur* (Leipzig), March 15, 1922; *Der Eisenbau* (Berlin), July 25, 1922.

TABLE 30.—Time Tests.

Test No. 22*a*. Temperature—20° Cent. (68° Fahr.) Fill is 5 ft. high, Top surface at + 31°. Vertical wooden wall.					Test No. 22*b*. Temperature—20° Cent. (68° Fahr.) Fill is 7 ft. 9 in. high against a vertical wooden wall 6 ft. high.				
Age of fill, in hours.	H_1	Total Pressures, in Pounds. *H.*	Total Pressures, in Pounds. *V.*	Weather.	Age of fill, in hours.	H_1	Total Pressures, in Pounds. *H.*	Total Pressures, in Pounds. *V.*	Weather.
0	834	2 504	1 458	Clear.	0	1 100	2 962	2 033	Clear.
¼	842	2 539	1 509	"	1	1 114	3 021	2 036	"
½	844	2 545	1 522	"	1¼	1 108	2 991	2 013	"
¾	846	2 552	1 532	"	43¼	1 088	2 960	2 004	Rain.
1	845	2 552	1 535	"	67	1 079	2 890	1 987	"
1¼	845	2 551	1 536	"	73	1 098	2 922	1 993	Cloudy.
1¾	845	2 554	1 541	"					
2	844	2 552	1 541	"					

Conclusions from this test are as follows:

The maximum pressure occurs soon after filling. The fill undergoes a settling as soon as it is placed, causing an increase in pressure in about 1 hour. The pressure then slowly decreases, reaching a low point within 24 hours. It then oscillates each day, as shown in the previous tests.

XIV.—Effect of Static Loads on the Lateral Pressure of Horizontal Fills against a Vertical Wall.

Forty-five square boards, 11¾ by 11¾ by 1 in., were used to cover the surface of a 6-ft. horizontal fill of sand. A keg of nails weighing 100 lb. was placed on each square, starting with the one farthest from the wall. Readings were taken while the load rested on the fill and after it had been removed. The effect was that of a localized surcharge of 100 lb. per sq. ft. The averages of the five positions of the load at each distance back of the wall gave the values shown in Table 31. The readings for the five positions of the load at each distance back of the wall were about the same. The surprising feature of this test was, that after the load had been removed, the pressure did not diminish immediately. The fill returned to its original condition several days later.

The theoretical plane of rupture, $\phi = 36°$, cuts the top surface at 3.06 ft. The plane of slope cuts the top surface at 8.25 ft.

Conclusions: There is an appreciable increase in pressure due to loads not on the wedge of rupture. The increase is much more rapid, however, as the load moves on the theoretical wedge. The effect of a static load is to change the physical characteristics of the fill, as the changed pressure continued after the load had been removed. Assuming that the unit weight has not been changed throughout the fill, and is 100 lb. per cu. ft., the final horizontal component requires a ϕ of 35° 10′. Assuming that the concentrated load compresses the entire fill to $\frac{7}{6}$ of its former weight, or 116.7 lb. per cu. ft., ϕ is then 38°40′. Assuming that the final density is the average

of these two extreme values, or 108.3 lb. per cu. ft., ϕ is then 37°00′. The formula used is $H = \frac{1}{2}\, y\, h^2 \tan^2 \frac{1}{2}\,(90 - \phi)$. The experimental value for ϕ of the material used is about 37°. It is probable, however, that both the density and the coefficient of resistance of the fill are changed.

TABLE 31.—TEST NO. 23.

Date: May 6, 1921.
Weather: Clear.
Temperature—15° Cent. (59° Fahr.)

Physical Properties: same as in Test No. 1.
d = distance of surcharged load of 100 lb. from back of wall.

Readings are the averages of five positions of load.

Distance, d, in feet.	PRESSURES PER FOOT, IN POUNDS.			Tan $\phi' = \frac{V}{H}$	$\frac{x}{h}$	PERCENTAGE CHANGES.				
	$H.$	$V.$	$P.$			$H.$	$V.$	$P.$	$\frac{V}{H}$	$\frac{x}{h}$
No load	469.2	286.8	551.6	0.610	0.367	.....	.....	.....	.....	.....
8.5	472.5	288.6	553.6	0.610	0.367	+0.7	+0.6	+0.4	0	0
7.5	473.4	288.9	554.6	0.611	0.367	0.9	0.7	0.5	+0.2	0
6.5	474.4	289.4	555.8	0.611	0.367	1.1	0.9	0.8	0.2	0
5.5	475.3	290.0	556.8	0.611	0.368	1.3	1.1	0.9	0.2	0.3
4.5	476.3	290.5	557.9	0.611	0.367	1.5	1.3	1.1	0.2	0
3.5	478.1	291.7	560.1	0.611	0.368	1.9	1.7	1.5	0.2	0.3
2.5	479.7	293.8	562.7	0.612	0.368	2.2	2.4	2.0	0.2	0.3
1.5	483.1	297.5	567.6	0.617	0.380	3.0	3.7	2.9	0.3	3.5
After test	483.5	299.3	586.6	0.620	0.380	3.0	4.5	3.1	1.6	3.5

The present method, used in practice to take account of static surcharges, assumes that ϕ is not changed, but that the density is changed. It requires that the readings, for a concentrated load at any point on the wedge, be the same. The test does not show this to be the case. It was impossible to place the load closer than 1 ft. to the wall. The results clearly show that the effect of a load depends on its distance from the wall.

XV.—Effect of Moving Loads on the Lateral Pressure of Horizontal Fills Against a Vertical Wall.

After completing the previous test, an attempt was made to measure the effect of loads rolling across the fill, parallel to the wall. The keg of nails was rolled three times across the width of the fill, at various distances from the wall, starting at the greatest distance. Readings were taken directly after this rolling, the load having been removed from the fill. The previous test had compacted the fill fairly well, so that the effect was that of a moving load on a settled or tamped fill.

Rolling or moving loads on a tamped fill have no effect on the pressure. In order to investigate the action of this compressed fill as it aged, it was allowed to stand for seven days. The summary given in Table 32 of pressures exerted by this fill is of interest. The height of fill was 6 ft., original $y = 100$ lb. per cu. ft., $\phi = 36°$.

The fill returns almost to its original state, except that the resultant rises 6.8 per cent. The decrease in the vertical component is due to the drying

out of the sand along the wall. The maximum at 96 hr. is due to a slight rupture in the fill. It is interesting to note that this packing and loading of the fill caused a variation of only 8.9% in the horizontal component, which is less than the variation in a similar fill caused by a 13°C. change in temperature over a like period of 166 hours.

TABLE 32.—Test No. 24.

Date: May 6, 1921. Physical Properties: Same as for test 23.

The pressures given are per foot of width of wall exerted by a 6-ft. horizontal fill against a vertical wood wall, after a load of 100 lb. has been rolled across the fill at a distance, d, from the wall.

Distance. d, in feet.	Pressures, per Foot, in Pounds.			Tan $\phi' = \frac{V}{H}$	$\frac{x}{h}$	Percentage Changes.		
	H.	*V.*	*P.*			*H.*	*V.*	*P.*
No Load	483.6	299.4	568.6	0.620	0.369	....	....	....
8.5	484.4	299.8	569.6	0.620	0.368	+0.2	+0.1	+0.2
7.5	484.6	300.0	570.0	0.619	0.368	0.2	0.2	0.2
6.5	484.6	300.1	570.0	0.620	0.368	0.2	0.2	0.3
5.5	485.0	300.1	570.4	0.619	0.369	0.3	0.2	0.4
4.5	485.4	300.4	570.8	0.619	0.369	0.4	0.3	0.5
3.5	485.8	300.6	571.2	0.619	0.369	0.5	0.4	0.5
2.5	485.9	300.9	571.4	0.619	0.369	0.5	0.5	0.7
1.5	486.6	301.6	572.4	0.619	0.370	0.6	0.7	1.0

TABLE 33.

	Pressures, per Foot, in Pounds.			Tan $\phi' = \frac{V}{H}$	$\frac{x}{h}$	Percentage Changes over Original Values.			
	H.	*V.*	*P.*			*H*	*V.*	*P.*	$\frac{x}{h}$
Just after filling	458.4	288.6	541.6	0.652	0.367	......	......	......	......
48 hr. later	469.2	268.8	551.6	0.611	0.367	+2.4	−0.6	+1.8	0
After Static Test	483.5	299.3	568.5	0.620	0.380	+5.5	+3.7	+5.0	+3.5
1 hr. later	483.6	299.4	568.6	0.620	0.369	+5.5	+3.7	+5.0	+0.5
After Dynamic Test	486.6	301.6	572.4	0.619	0.370	+6.2	+4.5	+5.7	+0.8
96 hr. later (144 total)	499.0	287.0	575.6	0.575	0.384	+8.9	−0.1	+6.3	+4.6
118 " " 166 "	464.8	268.2	537.0	0.577	0.392	+1.4	−7.1	−0.8	+6.8

Maximum values occurred at 96 hr. (total 144 hr.)
Minimum " " " 118 " " 166 "

XVI.—Effect of Static Loads on the Lateral Pressure of Horizontal Fill Against a Wall with Positive Back-Batter of 1:4.

A test similar to that in Section XIV was made with a 4-ft. fill and a wall sloping at + 1:4. The distance of the load is measured from the top of the wall. The heel of the wall was 1 ft. closer to the back of the bin; so that, in the last position, the load was directly above the heel of the wall.

The results of this test are similar to those obtained with a vertical wall. The change in vertical component disappears soon after the load is

removed, whereas the horizontal component remains undiminished. In the last position, the 100-lb. load is directly over the wall, yet the increase in total vertical component is only 17 lb.

TABLE 34.—Test No. 25.

Date: December 4, 1921.
Weather: Clear
Temperature = 3½° C. (38° Fahr.)

Physical Properties same as in Test No. 17.
Height of fill = 4 ft.
Wall batter = + 1 : 4 (14° 00′).
Load = 100 lb. per sq. ft.

d = distance in feet of load from top of wall.

Distance, d, in feet.	Total Pressures on Wall, in Pounds.		Tan $\phi' = \frac{V}{H}$	Percentage Increases:	
	H.	V.		H.	V.
No Load	823	746	0.907	0	0
8	823	746	0.907	0	0
7	823	748	0.909	0	+ 0.3
6	823	748	0.909	0	0.3
5	823	748	0.909	0	0.3
4	825	748	0.908	+ 0.2	0.3
3	828	748	0.905	0.6	0.3
2	831	756	0.910	1.0	1.3
1	840.5	761	0.907	2.1	2.0
No Load	840	763	0.908	2.1	2.3
1 hr. Later	840	747	0.890	2.1	0.1

General Conclusions on the Pressure of Sand Fills, Deduced from the Cincinnati Experiments.

1.—The fill does not act like a liquid. The transmission of pressure does not obey the Pascal law.

2.—The fill does not act like an elastic solid. Loads on the fill cause a change in physical properties as well as a deformation. These changes slowly disappear, but not according to any definite law.

3.—There is no sharply defined wedge of rupture, nor can any surface of rupture be detected on the surface of the fill. However, loads on the fill beyond the wedge of rupture have practically no effect on the lateral pressure. Loads on the wedge of rupture do not increase the pressure by as much as would be required by a well-defined and distinct wedge.

4.—The lateral earth pressure, for all types and kinds of wall and for all types of loading and fill, acts at an angle to the normal to the wall equal to the angle of static friction between the fill and the back of the wall. The height of application is above the $\frac{1}{3}$ point, but below the $\frac{4}{10}$ point of the height.

5.—For a vertical wall with horizontal fill, the value of the horizontal pressure is given by the formula, $E = \frac{1}{2}\, y\, h^2 \tan^2 \frac{1}{2}\,(90° - \phi)$, in which ϕ is the angle of internal resistance of the fill, corresponding to the static friction of the filling material on itself. This formula is a special case of both the Rankine and Coulomb theories. The value of the vertical component is easily determined from a consideration of Conclusion 4.

6.—For all general cases, the pressure is obtained closest by the general wedge theory

$$E = \tfrac{1}{2}\, y\, h^2 C; \qquad C = \left(\frac{\cos(\phi - \alpha)}{(n + 1)\cos\alpha}\right)^2 \frac{1}{\cos(\phi' + \alpha)}.$$

$$H = E\cos(\phi' + \alpha); \qquad n = \sqrt{\frac{\sin(\phi + \phi')\sin(\phi - \varepsilon)}{\cos(\phi' + \alpha)\cos(\alpha - \varepsilon)}}.$$

The wedge theory that disregards wall friction gives too high results except for a vertical wall, in which case it gives results closer to the experimental results than the general wedge theory.

7.—The general wedge theory gives low results for a vertical wall, and it is better to use the wedge theory, disregarding the wall friction,

$$H = \tfrac{1}{2}\, y\, h^2 N; \qquad N = \left(\frac{\cos(\phi - \alpha)}{(n + 1)\cos\alpha}\right)^2 \frac{1}{\cos\alpha}$$

$$E = H\sec\phi'; \qquad n = \sqrt{\frac{\sin\phi\sin(\phi - \varepsilon)}{\cos\alpha\cos(\alpha - \varepsilon)}}.$$

8.—The experimental results are not in agreement with the Rankine theory, except as noted in Conclusion 5.

9. The effect of settling is to increase the pressure exerted by the fill, the maximum being attained within two hours of placing the fill. The pressure then decreases to the minimum within 24 hours, after which the pressure oscillates toward equilibrium between the maximum and minimum thus found.

10. The pressure is a direct function of the temperature.

11. The pressure is only slightly affected by humidity, except that in dry weather evaporation of moisture from the fill along the wall, near the surface causes a greater horizontal and a less vertical component.

12. Surcharges, both static and dynamic, compress the fill and increase the resultant pressure. Such an increase disappears in time, usually in about seven days.

Résumé of Previous Experiments.

In the early literature on the subject of Lateral Earth Pressure, little is said about experiments, although many of the contributors to the theoretical development probably made some tests. Belidor, in 1720, stated that "experiment leads him to assume a 45° prism of rupture".* The earliest description of an apparatus is that of Gadroy who in 1745 tried to check Belidor's assumptions.† Although his experiments in a bin 3 in. high showed a plane of rupture at a slope of 2 : 3 for a sand with a natural slope of 1 : 1, which is an experimental proof that the plane of rupture is not the plane of repose, the only conclusion drawn was that a wedge of rupture, moving on the plane of slope, did exist. The early method of testing was to construct a box with one side closed by a gate, hinged at the bottom. The top of the gate was restrained by a cord, passing over a fixed pulley, tied to a pan. Weights placed in the pan balanced the overturning effect of the pressure on the gate. It is,

* Belidor, "La Science des Ingenieurs", 1st edition, 1729.

† Mayniel, "Poussee des Terres", 1808.

therefore, not surprising that a vertical component was not detected. It is surprising how many conclusions the experimenters did not notice. For example, Gauthey, in 1785, using a wedge-shaped box with the measuring gate vertical, obtained data showing that the pressure is constant for angles of the wedge between 22½° and 67½°, the material being sand. This proves that the wedge of rupture does not include all the material above the plane of slope, and yet, although Coulomb had issued his theory of the wedge of maximum pressure in 1773, Gauthey starts his theoretical discussion with the assumption of a wedge sliding on the plane of slope.*

The danger of obtaining the passive pressure and the method of overbalancing the gate to insure that the active pressure is being determined was first discussed by Woltmann in 1791. With a test-gate 4 ft. square his results were much lower than required by theory. The discrepancy would have been accounted for if he had taken into account the effect of a vertical† component. Experiments by Mayniel in 1806 and by deKoeszegh in 1828 showed similar results.

As early as 1800, Coulomb's contributions were misinterpreted. Hagen comments on this, emphasizing that Coulomb had called the friction coefficient of earth $\frac{1}{n}$ and had made no mention of any relationship between this coefficient and the natural slope. It was Woltmann who first made the assumption that the coefficient of internal friction was the tangent of the angle of repose. Unfortunately, he used this assumption in his German translation of Coulomb's Essai, and it is from this translation that later writers copied "Coulomb's Theory".‡

Hagen repeated Woltmann's experiments, and discovered that if he assumed a resultant inclined at a constant angle to the wall, there was perfect agreement between theory and experiment. The full-scale walls built and tested by Lt. Hope in 1845 showed results similar to those obtained from the Cincinnati tests.§ A wall, 23 in. thick, failed under the pressure of 10 ft. of fill, the material being ballast, weighing 95 lb., per cu. ft., and having a natural slope of 37°. The stability moment of the wall is 1 920 ft.-lb. and the overturning moment of the fill is 3 970 ft.-lb., disregarding the vertical component, as was done by Sir Benjamin Baker‖ in his discussion of Hope's tests. If the vertical component is taken into account, the overturning moment of the fill is 2 240 ft.-lb. Baker's contention that the wedge theory gives a factor of safety of 2 (*i. e.* 3 970 : 1 920) is not true, the overturning moment is slightly greater than the resisting moment.

Gen. Burgoyne, in 1834, built four 20-ft. walls, the average width of each being ⅓ the height, and tested them to destruction. No attempt was made to compare the results with theory; the application of the Cincinnati results gives the following:

* Gauthey, "Ouvres", Vol. I, edited by Navier, 1809.

† Woltmann, "Hydraulische Arkitektur", Vol. 3 and 4, 1794–9.

‡ Hagen, *Annalen der Physik und Chemie*, Vol. 116 (1833), pp. 17–48, 297–323; *Berichte* der k. Akademie der Wissenschaften (1853), pp. 35–42.

§ Hope, London Royal Engineers Papers, Vol. 7 (1845), pp. 69–86.

‖ Baker, "Lateral Pressure of Earthwork," 1883.

Wall A: both faces sloping 1 : 5, held a 20-ft. fill weighing 87 lb. per cu. ft. during heavy rains. M_r: 34 200, M_0: 14 900.

Wall B: vertical back, front battered at 1 : 5, held a 20-ft. fill after tipping about 2½ in. M_r: 32 400, M_0: 17 350.

Wall C: vertical front, back battered at 1: 5, failed by bulging and bursting at ⅓ the height when the fill reached a height of 17 ft. There was no tendency to overturn. M_r: 17 000, M_0: 6 000, failure due to low shear strength.

Wall D: both faces vertical, failed by overturning when the fill reached a height of 17 ft. M_r: 15 500, M_0: 17 500.

The result of each test agrees closely with the Cincinnati results. Because the fill was saturated, it was assumed that the unit weight was 100 lb. per cu. ft., the natural repose, 34°, and that the resultant acted at an angle of 34° to the normal, the walls being of rough rubble, laid dry.*

Aude, in 1848, obtained results about 85% of the values obtained by Coulomb's theory.† Considère, in 1870, showed that the results agreed very closely with theory, by taking into account the wall friction.‡

Winkler, in 1863, found a resultant always inclined at an angle equal to the angle of friction on the wall. To correct his results for the side-wall resistances, he repeated the tests with a thin partition wall in the center of the bin. From the difference in readings, he obtains the retarding effect of a side wall. This method would be reliable if the width of the bin was so great that there would be no chance of arching action in the narrow bins. By the use of two axes of rotation, one under the inner face of the wall and the other at the toe, he obtains the vertical and horizontal components, because in the first reading the vertical component is eliminated and the second shows the effect of both components.§ The objection to this method is, that in order to determine the total pressure, two independent tests must be made, and no account can be taken of changes in the conditions, which must be identical for accuracy. The experiments of Constable in 1874‖ and of Darwin in 1877¶ were made on so small a scale that the results are unreliable.

Leygue in 1885 reported a very complete set of experiments covering every phase of the earth pressure problem. He proved experimentally the co-existence of friction and cohesion in soils, an assumption made by Coulomb but not accepted by some of the later scientists. By the use of a bin with glass walls he investigated the shape of the wedge of rupture. He discovered that the surface of rupture varied but little from a plane, and that its shape was independent of the height of the fill, the method of wall failure (rotation or translation), of the nature or the slope of the wall and of the surface slope of the fill. His quantitative results are not so valuable because of the small

* Owen, *Transactions*, Irish Inst. Civ. Engrs. (1845), Vol. I, pp. 138–44.

† Aude, Memorial de le Officier du Genie, Vol. 9.

‡ Considère, *Annales des Ponts et Chaussées* (1870), Vol. 19, p. 547.

§ Winkler, *Der Civilingenieur* (1865), Vol. II, pp. 1–11; "Neue Theorie des Erddrucks" (1872) (not obtainable).

‖ Constable, *Transactions*, Am. Soc. C. E. (1874), Vol. 3, pp. 67–75.

¶ Darwin, *Minutes of Proceedings*, Inst. C. E. (1883), Vol. 71, p. 350.

sized apparatus. Winkler's method was used to determine the effect of the side walls.*

A large apparatus was constructed at the University of Nebraska by A. A. Steel in 1899, a test wall $14\frac{1}{2}$ ft. high being inserted in a pit. The horizontal and vertical components on two 12-in. square sections of the wall were measured by levers. For such a depth the pit was too small in cross-section and the effect of the solidified earth walls caused inconsistent readings. The labor involved prohibited more than a few tests and some of these were performed in parts. Steel noticed the drop in pressure over night, which he attributed to interference with the apparatus by troublesome boys. The same drop in pressure was noticed in the Cincinnati tests, in which the apparatus was safe from intrusion.†

The Mueller-Breslau apparatus, on which the Cincinnati design was based, was "an attempt to reproduce the actual action of a retaining wall under earth pressure". A complete translation of the section of Mueller-Breslau's book dealing with his apparatus and experiments is included in the writer's thesis, filed in the Library of the University of Cincinnati. Mueller-Breslau determined the horizontal and vertical components by measuring the deformations in calibrated steel struts, restraining a wall 29 in. high. No attempt was made to measure the effect of the side walls, which were coated with graphite to reduce the friction. One noteworthy observation, which caused the use of graphite, was that the angle of friction of sand on these walls was 14° whereas that of sand on glass was 22°.‡

His conclusions were, that the measured earth pressure is somewhat greater than the value from Coulomb's theory. The writer, by using the angle of internal resistance in place of the angle of repose, has shown that very close agreement exists between Mueller-Breslau's results and the wedge theory. The surface slope had no effect on the inclination of the resultant. Using glass and emery cloth backing on the wall, he concluded that the resultant is always inclined at the angle of friction on the wall. Alternate loading and unloading of the fill caused the resultant to rise. The same results were obtained in the Cincinnati experiments.

A photographic investigation of the wedge of rupture was made in a glass box, about 2 ft. high. The wall could be displaced by either a translation or a rotation. It was found that a movement of 1 to 6 mm., had practically no effect on the shape of the surface of rupture, which was slightly curved down at the lower end, approximating Coulomb's theory very closely. With surcharged fills, the lower end of the surface of rupture was straight and the upper portion is curved. Mueller-Breslau announces that he has designed a much larger apparatus, but the writer has been unable to find any further information than that given in his book of 1906.

At the University of Pittsburgh, Professor J. H. Smith constructed an apparatus for the determination of the lateral pressure. As the maximum distance between the front and the back walls of the box he used was only

* Leygue, *Annales des Ponts et Chaussées* (1885), Vol. 10, pp. 780–1000.

† Steel, *Engineering News* (1899), Vol. 42, pp. 261–263.

‡ Mueller-Breslau, "Erddruck auf Stützmauern", 1906.

18 in., the height being 5 ft., the results are only applicable to storage bins. The use of these results to retaining wall design is equivalent to the assumption that earth pressure is the same as liquid pressure.*

Crosthwaite (1916-22) investigated the accuracy of Rankine's formula for penetration: $d = \frac{P}{y}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^2 = \frac{P}{y}\tan^4 \frac{1}{2}(90 - \phi)$, by using a plunger of $3\frac{1}{4}$ sq. in. bearing area. Substituting the penetrations found,† the value of ϕ can be computed. Comparison is then made between this ϕ and the natural slope. Sand, garden earth, cinders, ashes, and clay were used. It was found in general, that in materials loosely deposited, the angle of friction equals the angle of repose. On consolidation of the fill, this did not hold true. The results obtained are the combined effects of cohesion and friction; there is no necessity to consider these phenomena separately. Although Crosthwaite draws definite conclusions, it should be noted that in most of his experimental work, results were widely scattered, many readings being 50% from the mean. He considers this due to the small-sized apparatus, and says, "Model experiments are best * * * of the experiments made by previous investigators to investigate the lateral pressure of earth, those in which model walls were used are of the greatest value, but if the models are of any size, the experimental difficulties are almost insuperable." The writer agrees with this last phrase.

Experiments with sand ranged from dry sand to saturated sand (35% moisture), with a range in weight from 108 to 121 lb. per cu. ft. The computed value of $\frac{1 - \sin\phi}{1 + \sin\phi}$ was 0.0500 to 0.0601, corresponding to values of $\phi = 64° 27'$ to $62° 27'$. The sand acted like a solid, the plunger came to rest immediately. The observed natural slope was 52° 28'.

Garden earth, with a natural slope of 46° 12', gave experimental values for ϕ of 43°08' to 60°51' with an average value of 58°15'. Ashes and cinders acted as viscous solids, and required a long time to come to rest. The results obtained are summarized in Table 35.

TABLE 35.

Materials.	Angle of repose.	Maximum experimental value of ϕ.
Leighton Buzzard sand, damp	52° 28'	67° 48'
" " " dry	48° 04'	64° 47'
Bournemouth sand	47° 21'	65° 32'
Garden earth	46° 12'	60° 51'
Ashes and Cinders through ⅛-in. sieve	53° 48'	72° 59'
" " " 30x30 mesh	47° 30'	68° 08'

Experiments with clay were made with a bearing area of 1 sq. ft., and loads up to 12 tons per sq. ft. With mud, the area was 16 sq. ft., with loads up to 1.25 tons per sq. ft. These tests were made in the natural undisturbed

* Smith, *Proceedings*, Am. Soc. for Testing Materials, Vol. 15, Pt. 2, pp. 382–98.
† Crosthwaite, *Minutes of Proceedings*, Inst. C. E., 1916, Vol. 103, and 1920.

ground, and showed that the internal coefficient of friction decreases rapidly with an increase in pressure. There were great variations in the results.

Crosthwaite concluded that the Rankine formula holds true, but that ϕ must be taken as the angle of internal friction and not as the surface slope. For clay the penetration varies as the square of the load. Had the experimenter taken the trouble to determine the actual coefficient of friction in the material, he would have found that the angle of internal friction is always less than the natural slope, whereas, in these tests, the average angle of friction is 68° 21′, with a corresponding natural slope of 48° 24′, almost 20° more. Without doubt, the effect of friction along the sides of the bucket used as the container had considerable influence on these results.

In his 1920 report Crosthwaite gives the opposite conclusion for the same type of experiments. "The angles of internal friction from these experiments was much less than the angle of repose, and was not constant for any one material, but depended on its state of aggregation, whether it was put loosely together, shaken, or consolidated by tamping."

Tests with a small vertical wall, hinged at the base, in which the overturning moment caused by sand fills was measured by the tension in a string tied to the top of the wall, caused Crosthwaite to conclude that the pressures as calculated from the Coulomb and the Rankine formulas are too high, especially for surcharges. Walls calculated by the Rankine theory, using the value of the angle of repose for ϕ would have a factor of safety of $2\frac{1}{2}$ to 4.

In order to test the wedge theory, a false bottom coated with glued-on sand was inserted that could be set at any angle with the back of the wall, so that the pressure produced by any wedge could be measured. He found that the wedge theories that take into account the friction between the wall and the fill, give correct results for the wedge of maximum thrust as long as the wall is not surcharged, but that the calculated pressures are 30% too great for surcharged walls. These results, he states, made him think that the theory was wrong, and that the friction between the wall and the fill should not be considered, and that ϕ was really the angle of friction. In view of the result just stated, that the theoretical and experimental results did agree, it is difficult to understand how Crosthwaite obtained his conclusions. He calculated the value of ϕ required to satisfy the values obtained for the lateral pressure of a horizontal fill against a vertical wall. This value, 42° 20′, he called the angle of internal resistance. The natural slope of this material was 35°, or less than the angle of friction. With this new value for ϕ and disregarding the friction on the wall, "the maximum calculated pressures were practically identical with the observed."

The next experiment was intended to determine whether or not wall friction did affect the horizontal pressures.

A long box was sawed at the middle of its length, into two compartments by a vertical cut. One-half the box was fixed to solid supports and the other half was placed on rollers. When the box is filled with sand, the two compartments are urged apart by the pressure of the sand on the plane passing through the saw cut. By measuring the tension in a string that prevented the half of the box on rollers from being pushed forward, the pressure

on the vertical plane was determined. The part of the box on rollers was then turned end for end, so that the sand in the fixed compartment was held in position by the solid end of the box, the face of which was coated with glued-on sand. "This is the same as if it had been retained by a rough wall, and if the friction between the wall and backing affects the amount of the horizontal thrust the amount of tension now required to keep the rolling box in position should have been less than before." A large number of experiments showed that there was practically no difference, and so he concludes that "it is not correct to assume that the friction between the wall and backing can affect the amount of the horizontal thrust."

Disregarding the truth of the conclusion, the writer doubts the right to draw such conclusions from the tests. What was shown was that the thrust of a mass of sand on a theoretical vertical plane bounding the mass is the same as that on a vertical wall replacing the material on the other side of the theoretical plane, if the surface of the wall is of the same material as the filling material. Had the experimenter used a smooth wall and obtained the same results, the conclusions might have an experimental foundation.

The final conclusions drawn by the author from all his work are:

1.—That the plane of rupture may be a convenient mathematical assumption, but has no existence in the granular material dealt with—at least, he was unable to trace any evidence of it in his experiments.

2.—That the angle of repose is a physical constant that relates only to the surface, and is represented in the interior of the mass of sand by the angle of internal friction.

3.—That the angle of internal friction is not a physical characteristic constant for any one material, but varies with the state of its aggregation.

4.—That friction between the back of a wall and its backing does not affect the amount of the resultant thrust. The writer believes that "horizontal" component of the thrust is meant, because the apparatus used gives no measurement of the vertical component.

5.—That the wedge theory, which takes into account the wall friction and the angle of repose, though giving correct results when applied to a wall without surcharge, or with negative surcharge, breaks down completely when applied to a surcharged wall.

The writer desires to call attention to the statements concerning the relative value of the angles of repose and internal friction. Although both the articles in "Engineering and Contracting" for March 31, 1920, and "Engineering News-Record" for January 19, 1922, start with a statement that "the angle of internal friction from these experiments was much less than the angle of repose," all the data given in these and other articles by the author show the opposite. The other conclusions are remarkably close to those drawn from the Cincinnati experiments, although the experimenter's right to give them from the tests he performed is doubtful.

By using a rotating test-wall apparatus in a bin 7 ft. high, Fulton,* in 1920, measured the overturning moment of various types of fill. The magni-

* *Minutes of Proceedings*, Inst. C. E., 1920. Abstract in *Engineering and Contracting*, March 31, 1920.

tude of the resulting moment was measured by observing the extension of a bar fitted with a Ewing extensometer.

The results were compared with the various theories, and, using the conclusions given in the abstract, it was found that:

1.—The overturning moment on retaining walls due to the filling, as calculated from a general formula based on the wedge theory, is approximately the same as ordinarily exists when the filling is within the limits of these experiments, and when the inclination of the inner face of the wall either outward or inward is not greater than usually obtains in practice. In calculating theoretical values, it was assumed that the resultant acts on the inner face at a point one-third of the height of the fill from the bottom of the wall and that it makes an angle equal to the angle of repose with the normal to the wall.

2.—That in walls having a filling, the surcharge of which is not continuous and unlimited, the general formula requires modification. This modification can be made by reducing the sliding prism of irregular section to one equivalent triangular section from which the overturning moment can be deduced either by direct calculation or by a convenient graphical method. Under these conditions, the position of the resultant pressure may range from 0.33 to 0.364 of the height of the fill from the base of the wall.

For a vertical wall with a horizontal fill, the Rankine theory gave results for the overturning moment greatly in excess of the observed values, whereas the wedge theory gave results quite close to those obtained by experiment. For surcharged fills and a vertical wall, the Rankine theory gave values 50% in excess, and the wedge theory 20% in excess, of the observed values. Although three materials were used, clean river sand, gravel, and garden soil, it seems that most of the tests were made with sand. In order to keep a full plane of rupture inside the fill, surcharged fills were made smaller than the height of the bin, 7 ft., hence it may be concluded that the bin was rather narrow. With sloped walls the theoretical values were from 40% to 50% in excess of the experimental values.

Moulton* classifies all materials between liquids and solids, as

A—solid, from moist sand to rock, dealing with walls.

B—granular, from dry sand to wheat, dealing with bins.

C—semi-liquid, from wet clay to quicksand.

He dealt only with the first, and from many field observations concluded that the surface of rupture and surface of repose are not plane; that fracture in these materials is conchoidal. He notes that in one of the subway tunnels, the floor of which was 68 ft. below street level, a break in the surface occurred at a distance of 50 ft. from the center line; the average line of rupture being on a slope of about 1 : 2. In the placer mine gravel banks at Dawson, Yukon Co., Canada, the natural slopes averaged about 1 : 2, for heights up to 100 ft., and were always concave. He therefore concludes that the wedge behind a wall must exert pressure in accordance with Meem's theory.

It may be noticed that the writer makes no comment on the experimental work of E. P. Goodrich and J. C. Meem, Members, Am. Soc. C. E., and the

* Am. Inst. of Min. and Metallurgical Engrs., 1920.

discussions thereon, as published in *Transactions.* Some recent work by Finnan and Hummel, of the Institution of Civil Engineers (1921), in general agrees with the writer's results. As the work is still in progress, no more than mention is required. Terzaghi* rejects all the existing theories and starts the investigation of the stresses in granular fills following the method of Couplet (1727) but including the frictional forces between grains. The lateral pressure per unit area due to liquids is given by the well-known hydrostatic formula, $p = wh$. The same law holds true for a mass of smooth spheres of equal size. For any other assemblage of smooth spheres, the lateral pressure may be greater or less and is denoted by $k\,w\,h = p'$. Statical resistance is called the cause of the difference between p and p', because it exists independently of any frictional resistances acting at the surface of the spheres. The statical resistance is merely the effect of the weight of individual grains transmitted by their neighbors to the wall of the confining vessels.

Should the spheres become rough, the value of p' is unchanged as long as the wall does not yield. After a given yield, frictional resistance will be set up due to the roughness of the grains requiring that work be done in deforming the shape of the body. During the failure of a wall, one may distinguish between the first phase, the interval in which the frictional resistance is increasing but has not yet reached a maximum, and in which the statical resistance remains constant, and the second phase, during which the mutual positions of the grains change. In a detailed study, the elastic properties of the grains must also be considered.

The experimental work, described in "Engineering News-Record" of Sept. 30, 1920, consisted of several sets of tests, the object of each of which was to investigate one single point in the theory. Although several diagrams are given, no scale or dimensions are given. At the time this article appeared, the writer was working on his apparatus and, therefore, was interested in knowing the type of apparatus and method used by Dr. Terzaghi. In reply to a letter, the author states in part:

"The preliminary investigations made for my own experiments have shown that the intensity of the earth pressure is strongly influenced by the side of the bin, unless the width of the bin is at least two and a half times greater than the height of the backfilling. As shown by Donath (1891) the use of scales to measure the vertical component is utterly unreliable because of the inevitable vertical displacement of the wall. That is the reason why I adopted an indirect method for measuring the vertical component [several comments are here inserted on the design of the Cincinnati apparatus].

"My own experiments were made on a very small scale, the height of the retaining wall being 10 cm. (4 inches) only. I was not interested in the quantities but in the principle and in this case the size of the box makes no difference. The goal of my investigations was the determination of the elastic constants of the sand."

Remembering the size of the apparatus, the investigation covered the following problems:

1.—Earth pressure at rest—the ratio of lateral to vertical pressure was determined. The vertical pressure was applied by a testing machine on

* *Engineering News-Record,* September 30, 1920.

a mass of sand in a square frame (no dimensions given) one side of which was closed by a steel tape. Pressure on the sand caused this tape to press against a similar tape. A third tape was passed between these two and by pulling this third tape out, the frictional force on it was measured. No account is taken of the resistances of the sides of the frame, nor of the pressure lost in deflecting the inner tape before it began to bind on the measuring tape, although the distance between the tapes could be made very small.

2.—Horizontal Component.—The test wall (10 cm. high) rested on frictionless rollers, and was tied to a weighing scale with a cord passing through the fill. The connection to the wall was probably at the third point. Displacement of the wall was horizontal (sliding) and occurred by reducing the weights on the scale so that the active pressure was obtained. Coarse sand (2–3mm.) was used, because it is easier to produce a fairly homogeneous backfilling with a coarse sand than with a fine sand. To obtain an idea of the size of the apparatus, note that 40 grains of sand in a line will extend the entire height of the wall.

3.—Vertical Component.—By using a single roller under the wall, and using an indirect method, the author claims to measure the vertical component with only a horizontal displacement. The bottom of the front end of the wall is provided with a steel button resting on a ground-glass surface. The filling exerts a vertical component on the wall causing this steel button (placed very close to the bin) to bind on the glass. Weights are placed on the farther end of the base of the wall until the wall is balanced over the roller, which is at about the mid-point of the base. The computation of the vertical component requires the use of the horizontal component which was independently obtained. The author states "no pull is exerted in a vertical direction by the measuring device on the retaining wall". He neglects the process of counterbalancing which causes the face of the wall to move up a trifling distance, it is true, yet a vertical displacement does occur. The writer wishes to note that in all present-day methods, stress is determined by strain in the direction of the stress; in other words, it is impossible to measure a vertical force unless there is a vertical displacement.

4.—Passive pressures were obtained in the same manner as the active components, but measurements were made while the wall was being forced against the fill. As shown in the Cincinnati experiments, the amount of such movement must be considerable, to correspond to the amount of yield in the "second phase" (as termed by the author), before the passive pressure will occur.

5.—Observing inter-granular movements.—Sheets of aluminum were cut in the shape of sand grain sections and a study made of the manner in which these sheets assembled. Various other tests, measuring the lateral pressure at rest and during the second phase are given. During the first phase, the angle of friction on the wall being 34°, the resultant acted at angles of from 5 to 20° for various kinds of fill. The measured angle of inclination increased with the total pressure. During the second phase, the lateral pressure, for all cases, acts normal to the back of the wall.

In discussing the action of the sand during passive pressure, he also notes two phases. During the first phase, the resistance of the backfilling increases in direct proportion to the distance of compression until the frictional resistance which acts between the surface of contact of the sand grains assumes its maximum value. The second phase starts with inter-granular movements, the intensity of which increases more rapidly than the earth pressure decreases. These movements are confined to a definite wedge of the mass resting against the wall, which statement can be obtained by a study of the equilibrium of a mass of spheres. This "slip" occurs only if the angle between the forces acting on some surface (surface of rupture) and the normal exceed a certain value, which is the "angle of slip".

"The earth pressure against a perfectly rigid wall is fairly independent of the backfilling." For sand, its value is $\frac{1}{2}$ (0.42) $y\,h^2$, and the minimum value for any backfilling can be caused by a yielding of the wall, and the corresponding coefficient varies from 0.15 to 0.05. It should be noted that all these formulas include the density, in spite of the previous statement that the density is independent of the material. The conclusions are given for the sake of completeness, not because of their quantitative value. The writer makes many valuable openings for future investigation.

The writer wishes to call attention to the close agreement of the independent experimental work, when such work was on a reasonably large scale and resulted in consistent conclusions, with the results of the Cincinnati tests. He believes that the smallest sized apparatus for the accurate determination of lateral earth pressure should be at least 5 ft. high, as was recommended by William Cain, M. Am. Soc. C. E., in 1915. Fairly accurate readings can be obtained with a height of 3 to 4 ft.; a test wall 6 ft. high will give at least three accurate readings 1 ft. apart, from which reliable results can be obtained. The readings should be taken as directly as possible, friction dynamometers, springs, elastic gages, etc., require too much calibration. Platform scales have proved reliable, accurate, and sensitive. Experiments on miniature apparatus are of no use; no one claims quantitative results, and the qualitative results are not consistent with each other.

Greater attention must be paid to the properties of the material used in the tests. Physical, as well as chemical and geological, analysis, including the determination of all the properties of the soil, is necessary for any comparison with the work of others.

Summary.

From the Cincinnati experiments, together with the results of other experiments, the writer recommends the following rules for the determination of the lateral active earth pressure, to be used in the design of retaining walls and similar structures.

1.—The resultant pressure is inclined to the wall, deviating from the normal by an angle equal to the angle of friction between the fill and the wall.

2.—The resultant acts above the third point, and, for a heavy surcharge, as high as the $\frac{4}{10}$ point.

3.—The amount of the horizontal component is given closest by the wedge theory, taking as the angle, ϕ, the experimentally determined angle of internal resistance of the fill.

The effect of surcharge, temperature, settling, etc., is found at the end of the Cincinnati tests report.

The writer hopes to see the work at the University of Cincinnati continued, thereby completing the solution of the accurate experimental determination of the lateral pressure of granular materials. This paper concerns only the first of a series of tests that the writer planned, but could not complete. The labor required is too much for an individual. The University will welcome co-operation; the writer understands that the Society was asked to recommend a graduate student to fill the vacant Baldwin Fellowship in Civil Engineering. He hopes that the Society will avail itself of this opportunity to get in touch with the work, and he will gladly give whatever assistance he can to bring the problem to a successful solution. He wishes to thank the University for the opportunity of making these tests.

ELASTIC STRESSES IN THE ROCK SURROUNDING PRESSURE TUNNELS

By Charles P. Dunn,* Assoc. M. Am. Soc. C. E.

Synopsis

This paper is a theoretical study of the stresses occurring in the rock surrounding pressure tunnels, the purpose of which is to assist in developing pressure-tunnel design, with particular reference to that part of high-pressure power conduits and penstock tunnels in which the rock cover alone is insufficient to withstand safely the internal pressure. A method is outlined for making calculations to determine whether strength must be provided in the tunnel lining, and if so how much.

The subject is discussed as follows:

Part I, Theory, in which is presented a study of elastic stresses caused by internal pressure in a circular tunnel, the rock being assumed to be a homogeneous, weightless substance, with the modulus of elasticity and factor of lateral deformation as of rock.

Part II, Practical Application, in which is described a method of applying the results of Part I to tunnel design, the weight of the covering material being considered.

Introduction

This subject is of interest to many engineers because the large size of modern power installations makes the pressure tunnel the most economical form of conduit at many sites. A great number of such tunnels have been built recently, and many more are under construction or are contemplated.

The writer realizes that on account of the hazards and uncertainties of tunnel construction, pressure-tunnel design must always be controlled more by experience and common sense than by theory, but he believes that a consideration of the problem as presented in this paper will aid in forming a correct opinion.

In studying theoretically a problem of this kind, assumptions must be made, which are bound to be in error to a small extent and which will not fit exactly all conditions. The results of a study may be accurate mathematically, but the practical accuracy cannot be of greater refinement than the accuracy of the basic assumptions.

Note.—Written discussion on this paper which will not be presented at any meeting of the Society, will be closed with the **August, 1923,** *Proceedings*. When finally closed the paper, with discussion, will be published in *Transactions*.

* Designing Engr., Portland Ry., Light & Power Co., Portland, Ore.

The theory as presented takes no account of fault planes or stratification, the rock being assumed to have a uniform modulus of elasticity and factor of lateral deformation in all directions. For simplicity, all the calculations are based on circular bores and are, therefore, only roughly applicable to other tunnel sections.

The problem is treated as if the tunnel was a "thick cylinder" with an infinite external radius. This is sufficiently accurate for a practical study of the stresses due to internal pressure because, as will be shown, the great bulk of the stress is in the material immediately surrounding the bore, and only a small error is introduced by assuming the external radius to be infinite when actually it may be as small as 50 ft. over a tunnel 10 ft. in diameter.

I.—Theory

Consider first a tunnel of circular section in a weightless material, having otherwise the characteristics of homogeneous granite. Assume for the moment that the rock can carry tensile stresses elastically. Later, a reduction of compressive stress will be substituted for this tensile stress, which is assumed to behave in the same manner within the elastic limit, thus bringing the solution of the problem into the field of practical usefulness.

When fluid pressure is applied to the tunnel, the diameter will increase elastically until the internal pressure is balanced, the increase being a maximum at the wall of the tunnel and zero at an infinite radius. The deformation of the rock in diametral directions must occur in such a way that all its particles are in equilibrium. The successive annular rings will increase in diameter elastically until tension is set up in them, the direct radial stresses being gradually balanced by circumferential tension, until, at an infinite radius, all the stresses in radial directions will have been absorbed or balanced by circumferential tension.

Notation.—In this paper, L, R, and T denote stresses longitudinal, radial, and tangential, with respect to the axis of the tunnel. The subscript, a, denotes apparent stress, or stress external to the particle under consideration, and the subscript, t, the true or internal stress as measured by the deformation. Additional subscripts are used to locate the stress being discussed, such as R_{ax} to denote the apparent stress in a radial direction at the radius, x.

E = the modulus of elasticity;
λ = the factor of lateral deformation (Poisson's ratio);
e = the unit elongation;
r = the radius of circular tunnel; and
x or z = the radius to any point in the rock.

Referring to Fig. 1, consider a particle between r, an annulus at the wall of the tunnel, and z, another annulus of infinitesimally greater radius than r, the particle being enclosed between longitudinal and cross-sectional planes, and the cylinders, r and z. When pressure is applied to the tunnel, there will be a certain definite elastic increase in the radius of the cylinder, r, and another (smaller) elastic increase in the radius of the cylinder, z. The condition of equilibrium requires that the tangential and radial stresses

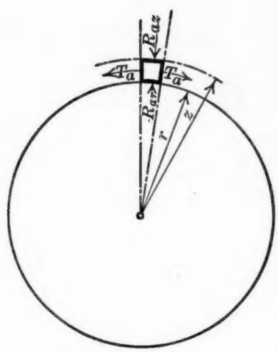

FIG. 1.

must adjust themselves so that the difference between these two increases in radius will be equivalent to the deformation caused by the mean true radial compressive stress between r and z acting through the distance, r-z, and the increase in radius at z must be the same as it would be if the cylinder, z, was the surface of a tunnel subjected to the unit pressure, R_{az}. R_{az} is less than the unit stress, R_{ar}, by the amount balanced by the circumferential tension between r and z, and it also is smaller than R_{ar} in inverse proportion to the length of the radii, r and z, because, as the particle under consideration is enclosed between radial (longitudinal) planes, the areas of its inner and outer surfaces are in proportion to the length of the respective radii.

Fig. 2 shows an infinitesimal particle of rock held in equilibrium by the three pairs of equal and opposite forces, T_a, R_a, and L_a. Equations (1), (2), and (3) are based on the theory of the true stresses.*

$$R_t = R_a - \lambda\, L_a - \lambda\, T_a \qquad (1)$$

$$T_t = T_a - \lambda\, L_a - \lambda\, R_a \qquad (2)$$

$$L_t = L_a - \lambda\, T_a - \lambda\, R_a \qquad (3)$$

It is apparent that the conditions in a tunnel are quite different from those in a gun or other finite "thick cylinder" which is free to deform longitudinally. It is assumed that there is no deformation of the rock longitudinally or parallel to the axis of the tunnel, because the tunnel may be considered very long in proportion to its diameter, and the thickness of the shell of the "thick cylinder" is very great in proportion to the diameter, therefore, the tendency toward longitudinal deformation is resisted by heavy shearing and bending stresses. It is realized that this assumption cannot be strictly correct, that there must be some deformation longitudinally, particularly near the ends of the tunnel, but the assumption of no longitudinal deformation appears to be much nearer the truth than that of absolutely free movement lengthwise as with a gun cylinder. Any intermediate consideration would introduce unwarranted complications in the problem, as the percentage of error involved in this assumption is small.

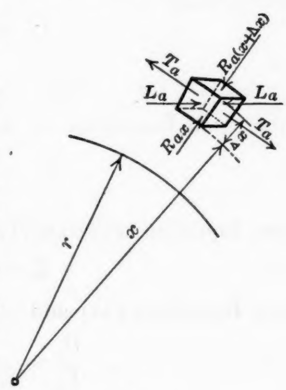

FIG. 2.

Then,

$$L_t = 0 \qquad (4)$$

* "Mechanics of Materials", by Mansfield Merriman, M. Am. Soc. C. E., 1908 edition, p. 360, Article 139.

and from Equation (3):

$$L_a = \lambda\, T_a + \lambda\, R_a \dotfill (5)$$

from Equations (1) and (5):

$$R_t = (1 - \lambda^2)\, R_a - (\lambda + \lambda^2)\, T_a \dotfill (6)$$

from Equations (2) and (5):

$$T_t = (1 - \lambda^2)\, T_a - (\lambda + \lambda^2)\, R_a \dotfill (7)$$

from Equation (7):

$$T_a = \frac{T_t + (\lambda + \lambda^2)\, R_a}{(1 - \lambda^2)} \dotfill (8)$$

from Equation (6):

$$T_a = -\frac{R_t - (1 - \lambda^2)\, R_a}{(\lambda + \lambda^2)} \dotfill (9)$$

from Equation (6):

$$R_a = \frac{R_t + (\lambda + \lambda^2)\, T_a}{(1 - \lambda^2)} \dotfill (10)$$

from Equation (7):

$$R_a = -\frac{T_t - (1 - \lambda^2)\, T_a}{(\lambda + \lambda^2)} \dotfill (11)$$

also, as true unit stress is proportional to deformation:

$$T_t = e\, E \dotfill (12)$$

As the modulus of elasticity and Poisson's ratio are assumed to be the same in all directions, it is reasonable to consider (for stresses within the elastic limit), that,

$$\text{Deformation} = R_a \times \text{a constant}$$

or,

$$e \text{ (unit deformation)} = R_a \times \text{a constant} \dotfill (13)$$

then, from Equations (8) and (12):

$$T_a = \frac{e\, E + (\lambda + \lambda^2)\, R_a}{(1 - \lambda^2)} \dotfill (14)$$

from Equations (7) and (12), or from Equation (14):

$$e\, E = (1 - \lambda^2)\, T_a - (\lambda + \lambda^2)\, R_a \dotfill (15)$$

from Equations (15) and (13):

$$\frac{R_a}{T_a} = \frac{(1 - \lambda^2)}{(\lambda + \lambda^2) + E\, C} = \text{a constant} \dotfill (16)$$

Referring to Fig. 3, in order that equilibrium may exist, the total of all the forces due to internal pressure acting perpendicular to a radius within the bore, *B*, must be equal and opposite to the total of all the forces acting perpendicular to the same radius outside the bore, *A*.

As the external radius is considered to be infinite, the curve of T_a, starting at a definite value at the wall of the tunnel, must diminish to zero at an infinite radius in such a manner that the area, *A*, between the curve and the *x*-axis, will be equal to the area, *B*.

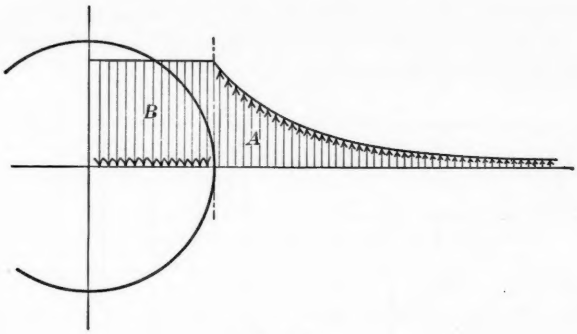

FIG. 3.

The equilibrium of forces may be stated in another manner, as shown in Fig. 4. Taking the forces perpendicular to any finite radius, x, the radial pressure on the annulus, x, is R_{ax}, and in order that equilibrium may exist,

$$R_{ax}\, x - \frac{2\; T_{ax} + \Delta\; T_{ax}}{2}\; \Delta\; x$$

must be equal to:

$$(R_{ax} - \Delta\; R_{ax})\; (x + \Delta\; x) \qquad (17)$$

which reduces to,

$$R_{ax}\; d\; x - x\; d\; R_{ax} = -\; T_{ax}\; d\; x \qquad (18)$$

The writer finds himself unable to write the equations of the stress curves by the use of calculus. He has used "cut and try" methods based on the relations established previously.

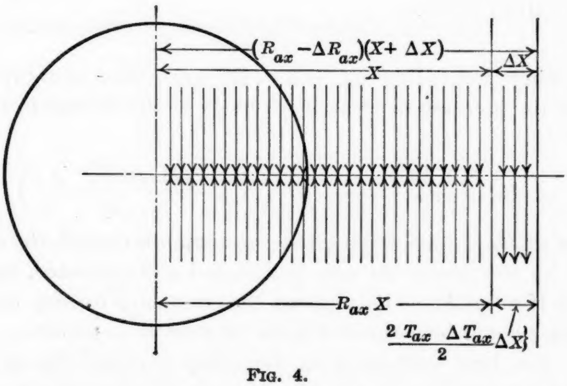

FIG. 4.

The method followed was the calculation of stress and deformation curves as shown in Fig. 5, by assuming, first, a value for the internal pressure, R_{ar}; and, second, an estimate of the corresponding value for the elastic increase in the diameter, $2r$. The curves were determined by calculating the stresses corresponding to small increments of radius successively. Starting at the wall of the tunnel, choosing a certain increment in radius, the mean elastic

increase in diameter and the mean radial unit stress were estimated for the increment. From these estimated values, Mean T_t was calculated by Equation (12), and Mean T_a by Equation (8).

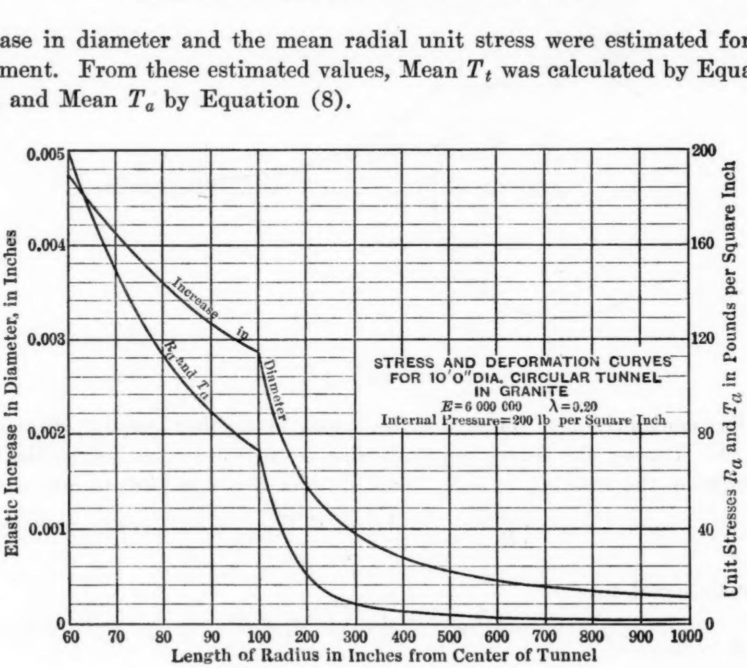

FIG. 5.

The radial unit stress at x, balanced by tension in the annulus of thickness, Δx, is, therefore, $\frac{\text{Mean } T_a \, \Delta x}{x}$, which is subtracted from R_{ax}. This remaining force, $R_{ax} - \frac{\text{Mean } T_a \, \Delta x}{x}$, is transmitted radially to the outside of the annulus to the point where the radius is $x + \Delta x$, the unit stress being reduced in direct proportion to the increase in area, or inversely as the change in radius. Therefore,

$$R_{a\,[x+\Delta x]} = \frac{x}{x + \Delta x}\left(R_{ax} - \frac{\text{Mean } T_a{}^{*} \, \Delta x}{x}\right) \dots\dots\dots\dots (19)$$

The value of $R_{a\,[x+\Delta x]}$ having thus been approximated, the estimated value of Mean R_a for the increment may be checked and corrected, and the calculation repeated if necessary. By plotting the curve step by step with the calculation, the values could be estimated quite accurately in advance.

Mean R_t was then calculated by Equation (6), and the deformation due to true radial compression in the annulus, Δx, is calculated by:

$$\text{Deformation in thickness, } \Delta x = \frac{\text{Mean } R_t{}^{\dagger} \, \Delta x}{E} \dots\dots\dots (20)$$

The increase of the radius, $x + \Delta x$, due to elastic stress is less than the increase of the radius, x, by the amount of compressive deformation in thick-

* Mean T_a denotes the mean between x and Δx.

† Mean R_t denotes the mean between x and Δx.

ness, Δx, in a radial direction. This latter was determined by Equation (20) and subtracted from the increase at the radius, x. The amount of the elastic increase in diameter at $x + \Delta x$ having thus been approximated, the value assumed for the mean could be checked, and corrected if necessary.

It was found that the curves were rather sensitive to errors in preliminary assumed values, too large an assumed value of elastic increase in diameter causing the curve, R_a, to cross the x-axis and too small a value causing the curve, T_a, to cross the x-axis, either of which conditions evidently cannot exist if all the conditions outlined previously are satisfied.

When the correct value of increase in diameter is assumed, the curves, R_a and T_a, coincide, indicating a value of 1 for the constant, $\frac{R_a}{T_a}$, in Equation (16), and approach the x-axis at an infinite radius.

A number of calculations have been made, with variations of λ, E, diameter, and internal pressure, to determine the effect of each on the result, and the following equation for the elastic increase in any diameter, $2x$, has been established beyond any reasonable doubt, and is sufficiently accurate for any practical calculation:

Elastic increase (of any diameter, $2x$)

$$= \frac{2(1+\lambda) R_r r^2}{E x} \quad \text{......................(21)}$$

Elastic increase (of diameter of tunnel, $2r$)

$$= \frac{2r(1+\lambda) R_{ar}}{E} \quad \text{......................(22)}$$

Equation (23), for the stress curves, R_a and T_a, was also established by the calculations:

$$T_{ax} = R_{ax} = \frac{r^2}{x^2} R_{ar} \quad \text{......................(23)}$$

Referring to Fig. 3, this calculation is checked by,

$$R_r r = \int_r^\infty R_{ax}\, d x = r^2 R_{ar} \int_r^\infty \frac{1}{x^2}\, d x \quad \text{............(24)}$$

Fig. 6 shows the elastic change in diameter, in inches, of a circular tunnel in rock for various values of λ, E, diameter, and internal fluid pressure, when the external radius is infinite.

II.—Practical Application

Referring to Fig. 7, consider, first, the forces holding the rock particles in equilibrium before the tunnel is excavated.

The vertical stresses, v, are imposed by the weight of the overlying material, and the horizontal stresses, h, are imposed by the tendency to lateral deformation. It may be assumed that, in a large body of rock, lateral deformation does not actually occur, because the horizontal forces in any given horizontal plane are equal and opposite in all directions. If this assumption is made, the true

or internal horizontal stress is zero, and an apparent or external horizontal stress exists in all directions in a horizontal plane which is measured by λv.

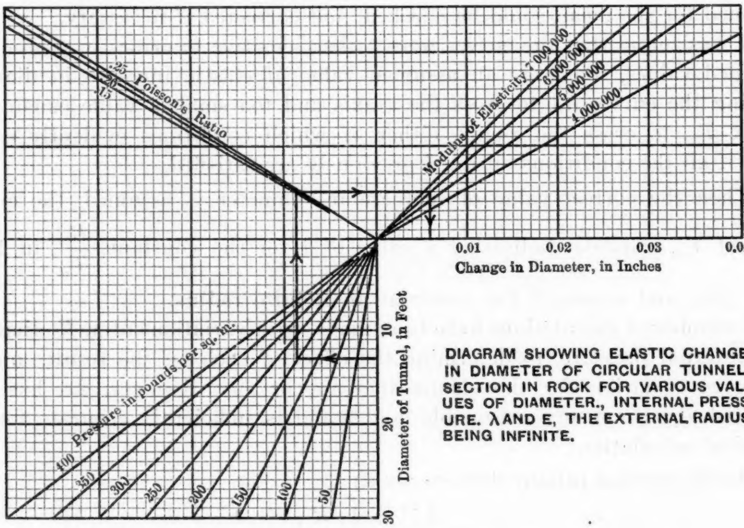

FIG. 6.

The true vertical stress is, therefore,

$$v_t = v_a - 2\,\lambda^2\,v_a = v_a\,(1 - 2\,\lambda^2) \ldots\ldots\ldots\ldots\ldots (25)$$

When the tunnel is excavated, it may be said that the struts, H and V, are removed, which brings about a change in stresses such that the loads formerly carried by these struts are carried by circumferential compression, and, within the elastic limit, the distribution of the additional stresses will be as presented in Part I, with opposite sign. The apparent circumferential compressive stress, T_a, at Point A, caused by the removal of the force, V, will be equal in amount to V; this will be added to the horizontal stress, H, already existing at A, making the total apparent tangential stress at A equal to $V + H$. In like manner (neglecting the difference in the quantity of material overlying A and B), the tangential stress at B is the same as at A, namely, $H + V$.

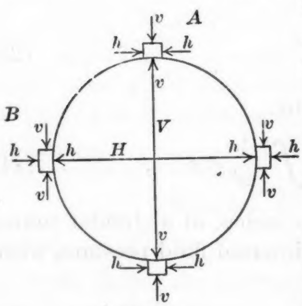

FIG. 7.

If internal fluid pressure is applied to the tunnel at a unit pressure, $H + V$, the initial circumferential compressive stresses will be balanced by the tendency of the internal pressure to produce tension. Any additional internal pressure will produce apparent circumferential tension in the rock, and there will be a tendency for the rock particles to separate, and for existing natural seams, relieved of initial compression, to open.

The writer believes that (with a safety factor applied) this method of calculating the strength of pressure tunnels may be of some value in tunnel design and has prepared Fig. 8 to show the relation of depth of rock over-

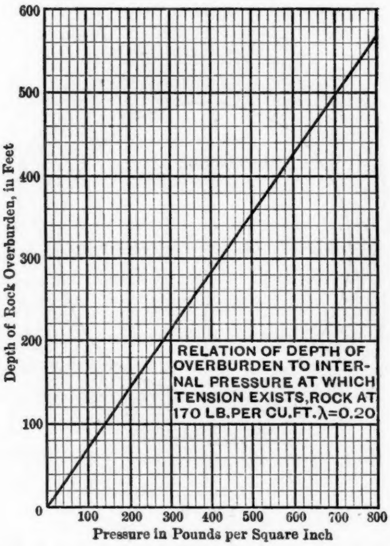

FIG. 8.

burden to the critical pressure at which apparent tension exists in the rock immediately surrounding the bore. It will be noted that no safety factor is included in the diagram. The factor of lateral deformation, λ, is taken as 0.20 and the weight of the rock at 170 lb. per cu. ft. Apparent tension exists whenever the head of water, in feet, is more than three and one-quarter times the head or the rock over-burden, in feet.

It must be borne in mind that any rock may contain seams, arched over, which will not be closed by the weight of the over-burden. Such seams exposed to water pressure would allow a dangerous and possibly disastrous hydrostatic pressure to develop over large areas in the rock. For this reason, pressure tunnels ordinarily must be sealed with a lining as nearly watertight as practicable. Where the over-burden is ample, the lining should be a dense concrete and the seams in the rock should be grouted under pressure. Where the tunnel approaches the surface as in the case of a penstock, the lining should be of steel plate of whatever strength may be necessary to make up the deficiency in the strength of the rock, with a backing of porous concrete freely drained.

The writer believes that concrete placed between steel lining and the rock in penstock tunnels should be a porous mix, with drains placed in it to carry away any leakage through the lining and to relieve external ground-water pressure. (See Fig. 9.) If such construction is impracticable, other provi-

sion must be made for protecting the steel against crushing by external pressure, such as angle or channel collars, or a ring of concrete inside the steel. No grout should be used in steel-lined penstock tunnels where porous concrete and drainage are practicable.

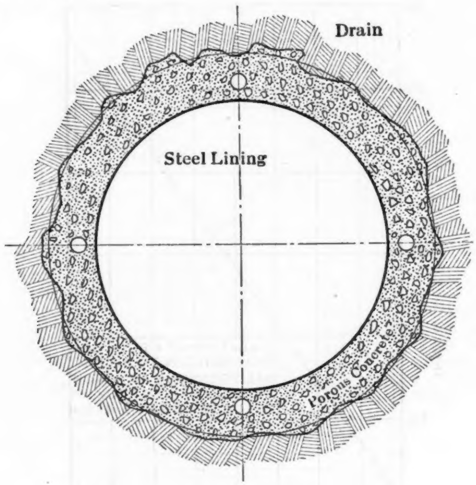

FIG. 9.

The concrete backing of a steel penstock tunnel lining need not have great strength, because (*a*) the shrinkage of the concrete on setting will be sufficient to render it ineffective, except to transmit direct stresses in radial directions (this makes the effective diameter of the tunnel the diameter of the excavation); and (*b*) ordinarily, the pressures to be transmitted in a radial direction are no greater than can be safely carried by a lean concrete.

The writer invites discussion on the subject and hopes that valuable practical experiences with pressure tunnels will be related therein.

A RIVER DIVERSION ON THE DELTA OF THE COLORADO IN RELATION TO IMPERIAL VALLEY, CALIFORNIA

By S. L. Rothery,* Assoc. M. Am. Soc. C. E.

Synopsis

This paper describes a diversion on the delta of the Colorado River in Baja California, Mexico. The reason for the diversion, the conditions that existed to affect its success, the considerations that determined the work undertaken, and the manner in which the diversion was completed, are discussed.

An initial stream flow that was only a small proportion of the flood discharge was used to start the diversion. The river made its own channel along the diverted route, eroding and transporting probably from 12 000 000 to 18 000 000 cu. yd. of soil, whereas, in the 4 miles of dredging operations, only 657 000 cu. yd. were excavated.

Historical

The courses of that part of the Colorado River in Mexico and of its delta effluents are wholly in its own alluvial silts which now replace what formerly was the northern end of the Gulf of California. From the present head of the Gulf, a great bed of alluvium extends north and includes the Salton Sea, in California. (See Fig. 1.) The Salton Sea is situated at what was formerly the northern end of the Gulf, and its bottom is about 275 ft. below mean sea level. The width of the alluvium bed is shown on Fig. 1, within the contour line marked "Ancient Beach Line."

In early ages, this turbid river deposited its silt burden in the narrow Gulf, building a deltaic ridge across it to the western shore and cutting off the northern end. The Colorado has since been distributing soil on this ridge, and the flow alternating down the slopes in cycles has produced depositions of stratified silt and alternate submergences and re-appearances of the valley bottom. When the flow was entirely down the Gulf slope, evaporation left the valley soils parched and arid.

Complete submergence to about 30 ft. above sea level, which is the elevation of the delta ridge at its western end, would have occurred again during the present century had not the agricultural possibilities of the valley led to the development, since 1900, of an extensive international irrigation project on which from 600 000 to 650 000 acres of this fertile alluvium are now irrigated. During 1905-06 the entire flow of the Colorado River was into

Note.—Written discussion on this paper, which will not be presented at any meeting of the Society, will be closed with the **August, 1923**, number of *Proceedings*. When finally closed, the paper, with discussion, will be published in *Transactions*.

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the valley, and the present Salton Sea was then formed. This inflow, its cause, and the operations undertaken to force the river back on its delta ridge (at the location marked *A* on Fig. 1), have been described.*

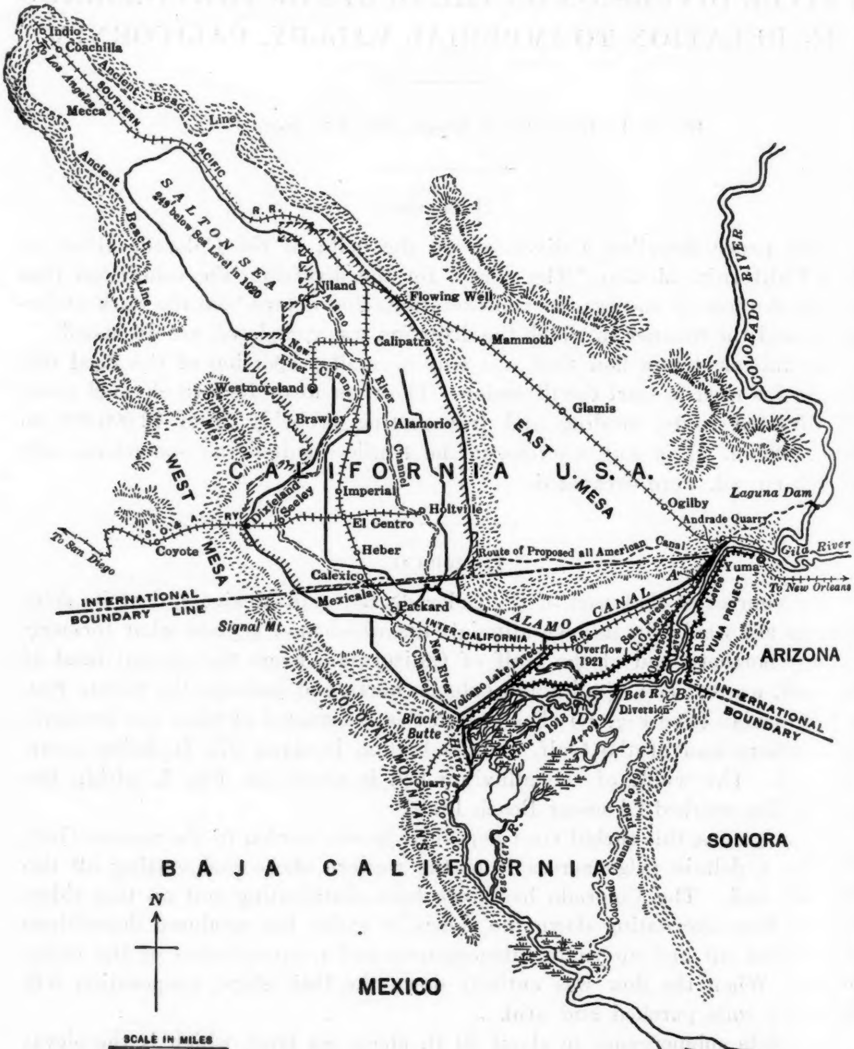

FIG. 1.

Since that outbreak, the tendency of the river has been to flow down the valley slopes and to abandon its course to the Gulf. The definite changes in the course of the river shown at *B, C,* and *D,* on Fig. 1, occurred in 1909,

* " Irrigation and River Control in the Colorado River Delta", by H. T. Cory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

1918, and 1921, respectively, and are in that direction. To oppose this tendency, the levee system shown, has been built. Following the operations that returned the river to its course to the Gulf, in February, 1907, the levee of the California Development Company was built to deflect the flood waters farther to the south. In the following three to five years, the Ockerson, Saiz, and Volcano Lake Levees were built to the heights then deemed necessary. All these levees, having a total length of 65 miles, have had to be heightened and strengthened, from time to time, and continually repaired at the breaches made in them during floods.

The western 7 miles of the Volcano Lake Levee has been raised 10.5 ft. since 1908. During this period, the elevations of the flood waters have increased each year, until in June, 1921, they were 13.7 ft. higher than those of 13 years ago. Prior to 1920, more than $900 000 had been spent on this levee alone. Both the Volcano Lake Levee and the Ockerson Levee have been heavily revetted with rock over their entire river slopes, and some engineers advocate further raising and strengthening to keep ahead of the growing elevations of the deltaic ridge and the consequent higher flood levels.

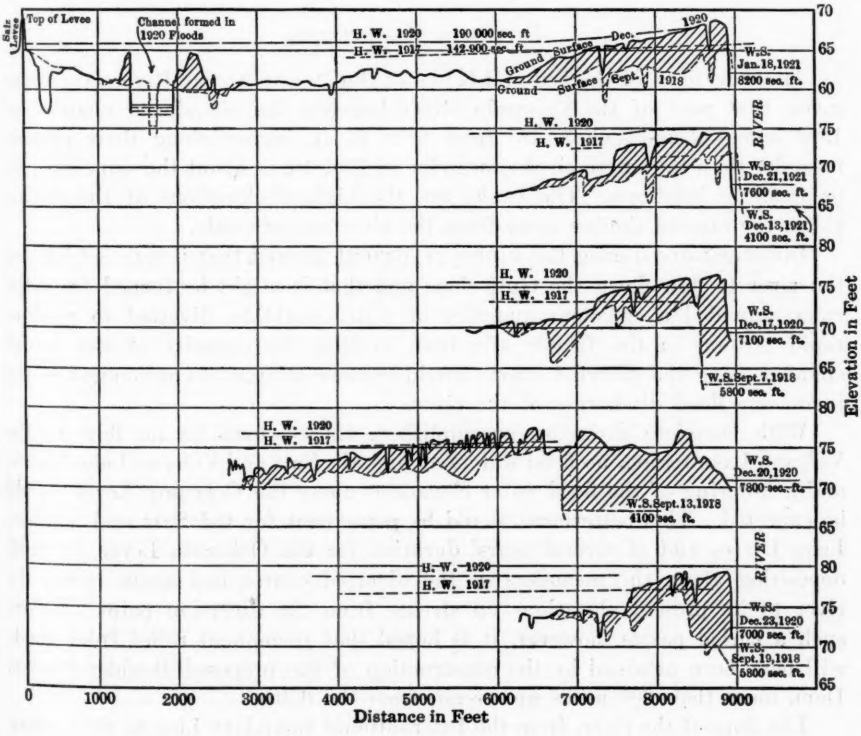

FIG. 2.

Fig. 2 shows the elevating process of the river bed and its banks. The shaded portion is the blanket of silt deposited on the north bank of Bee River between September, 1918, and December, 1920, at locations several miles

apart. During this period, only one flood of long duration occurred (June, 1920), that could have contributed most of this deposition. The flood water that overflows the river banks loses its velocity, due to spreading through the heavy growth of brush. The silt burden is deposited and the ground is thus elevated. Although sufficient channel capacity for a major part of the flow of the river is maintained, deposition tends to keep the river on the ridge, as each bank becomes, in effect, a thickened levee several hundred feet wide, well revetted with a dense growth of brush and trees through which the overflow can percolate but cannot erode channels in the slope. As the flood velocities diminish, the river bed is refilled with silt and sand-bars that deflect the flow of subsequent freshets, causing undercutting and caving of banks. These changes in the meandering of the river thus broaden as well as elevate the ridge.

The large annual expenditures for levee building and maintenance are burdens, and the weakened condition of the levees during high floods causes apprehension and anxiety which are harmful both to land values and to the entrance of capital.

Diversion Possibilities

The elevations of the bed and banks of the Bee or Abejas River (the name given that part of the Colorado River between the abandoned channel of 1909 and Volcano Lake), are from 8 to 15 ft. higher along their present meander than they were on the meander of 1909-10, at about the same easterly or westerly locations. The banks are the highest elevations of the deltaic ridge, the ground sloping away from the river on each side.

Investigations during the winter of 1920-21 showed that it was feasible for the river to flow down the Gulf slope and that it might be turned from the ridge provided a sufficient quantity of water could be diverted to produce rapid erosion in the friable alluvium, so that the capacity of the initial channel along the diverted course would become enlarged to accommodate the increasing flood discharges of the river.

With complete diversion accomplished, there would be no flow to the Volcano Lake region, no flood waters against the Saiz and Volcano Lake Levees, and a lowering of the flood-water elevations along the Ockerson Levee should be expected. This betterment would be permanent for the Saiz and Volcano Lake Levees and of several years' duration for the Ockerson Levee, or until deposition along the meanders of the diverted course had again raised the river to its present elevations up stream from the diversion point. Before such a period passes, however, it is hoped that permanent relief from floods will have been attained by the construction of the proposed Boulder Canyon Dam, more than 300 miles up stream from the delta.

The slope of the river, from the International Boundary Line to the western end of the Saiz Levee, near Volcano Lake, was taken by noting at 11 A. M., on December 21st, 1920, the water surface elevations at these two extreme points and at four intermediate points. The slopes varied between 0.85 ft. and 1.25 ft. per mile.

To determine the site for the diversion, surveys of both banks of the Bee River westerly from the Ockerson Levee were made. Profiles were also run across the south bank toward the old 1909 channel, which is visible in places, and toward the upper ends of deltaic effluents, about 7 miles westerly.

The site, shown on Fig. 1, was chosen because (1) it is the first point where the river is between defined narrow banks, thus affording the best feasible point at which to close the river channel; (2) the quantity of excavation would be less than one-half that required for similar channels directed toward the old 1909 channel; and (3) the meandering of the river, to the south and southeast of the Ockerson Levee, was so wide and changeable that the inlet to excavated channels might not remain connected with the main river.

During September, 1921, an aerial survey of the river between the International Boundary Line and Volcano Lake was made, that gave a more comprehensive view of the main channel with its effluent streams, than could have been obtained otherwise. The survey was not precise, but, from existing traverses and levees, the small errors could be corrected for mapping purposes. It covered an area of 214 sq. miles and cost $3 000, or $14 per sq. mile.

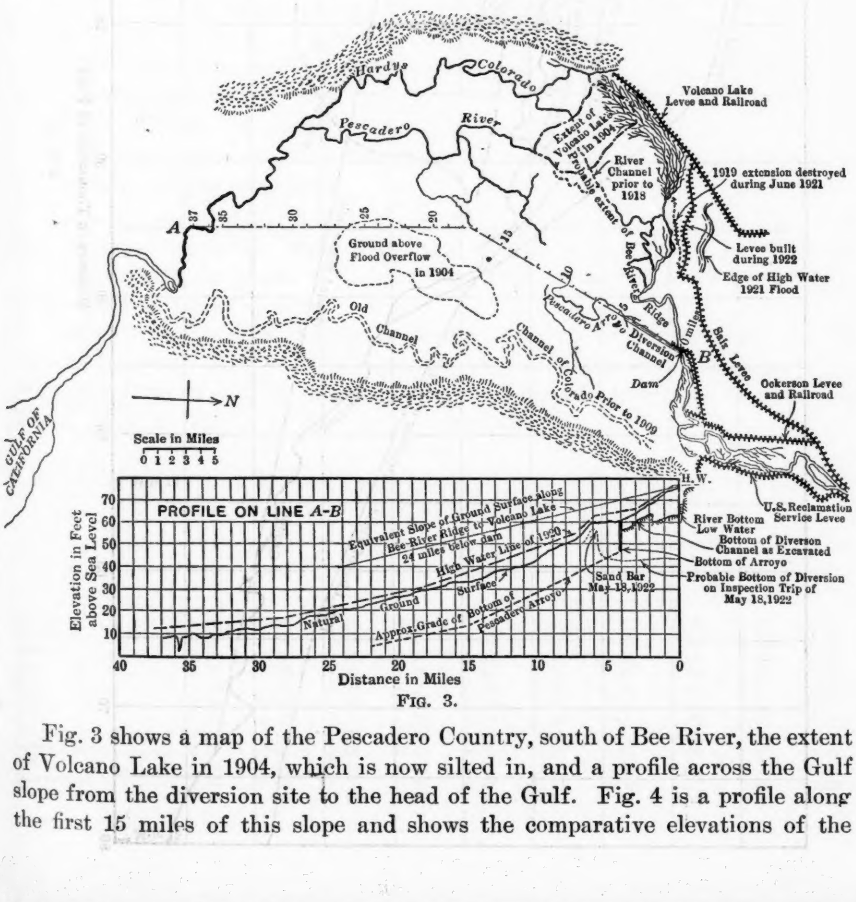

FIG. 3.

Fig. 3 shows a map of the Pescadero Country, south of Bee River, the extent of Volcano Lake in 1904, which is now silted in, and a profile across the Gulf slope from the diversion site to the head of the Gulf. Fig. 4 is a profile along the first 15 miles of this slope and shows the comparative elevations of the

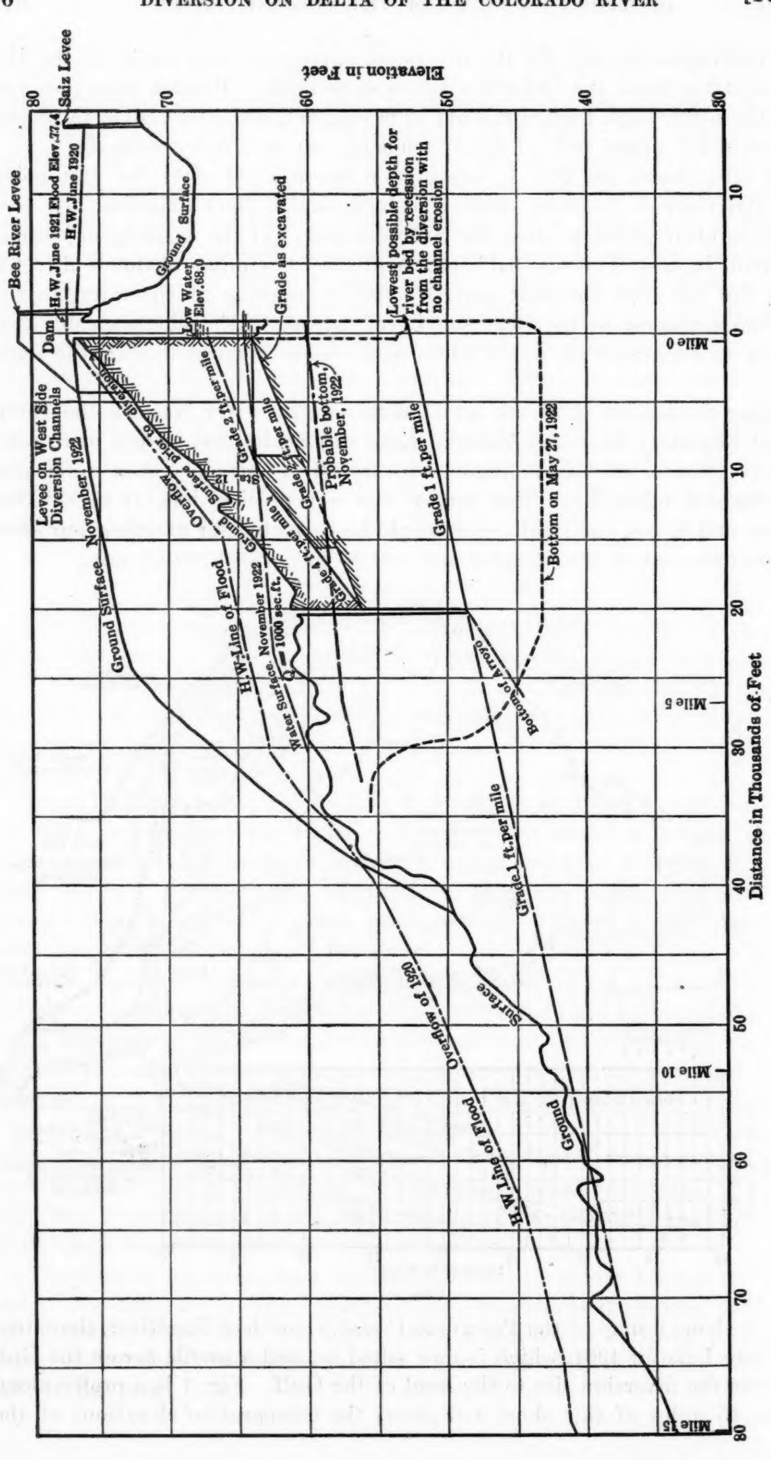
Elevation in Feet
Distance in Thousands of Feet
Bee River Levee
Saiz Levee
H.W. June 1921 Flood Elev. 77.4
H.W. June 1920
Dam
Levee on West Side Diversion Channels
November, 1922
Ground Surface
Low Water Elev. 68.0
Grade as excavated
Lowest possible depth for river bed by recession from the diversion with no channel erosion
Grade 2 ft. per mile
Grade 2 ft. per mile
Grade 4 ft. per mile
Probable bottom, November, 1922
Grade 1 ft. per mile
Bottom on May 27, 1922
Overflow
Ground Surface prior to diversion
H.W. Line of Flood
Water Surface. November 1922
Q = 1000 sec. ft.
Bottom of Arroyo
Grade 1 ft. per mile
Overflow of 1920
Ground Surface
H.W. Line of Flood
Mile 0
Mile 5
Mile 10
Mile 15

FIG. 4.

levees, high-water lines, and the height of the proposed dam. The grade lines of the bottoms of the proposed channels in the first 4 miles are also shown, together with the lowest gradient that should be expected, owing to erosion and bed recession becoming stabilized on a slope of about 1 ft. per mile, which is the grade apparently maintained in the alluvium for large volumes of flow. These profiles show the feasibility of a diversion from the location selected, which is enhanced by the existence of the effluents that serve to confine the diverted waters in channels and thereby to assist in draining the water away from the lower end of the dredged channels.

River Characteristics Affecting the Diversion

The following are the characteristics of the river and the conditions that exist at the site to affect the success of the diversion:

(1) High floods of 100 000 to 200 000 sec-ft., are certain in May or June of each year, and are probable in January or February. Freshets of 10 000 to 60 000 sec-ft. may be expected at any time.

(2) Low discharges of 4 000 to 10 000 sec-ft., for periods of several weeks' duration, occur between August and March.

(3) The river lacks channel capacity for its high discharges.

(4) The bed of the river varies in elevation at low and high discharges.

(5) Overflowed ground not subject to erosion produces a rapid growth of dense vegetation.

(6) The capacity of the river for soil transportation and deposition is great.

(7) For soil erosion, large quantities of water flowing at high velocities within channels are necessary.

Characteristics (6) and (7) are worthy of brief discussion. The soil transportation comprises the silt content carried in suspension at all times of the year and the sands or heavier silts that are rolled along the bottom of the river in times of flood.

Tables 1 and 2 give the mean average monthly percentages for the 10 years ending with 1921, and the maximum percentages in each month for 1919, 1920, and 1921, of silt in suspension in the Colorado River, as measured at Yuma, Ariz., by the U. S. Reclamation Service.

TABLE 1.—Monthly Mean Percentages of Silt (by Weight), in Colorado River Discharges for 1912 to 1921, Inclusive.

Month.	1912.	1913.	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.
January	0.28	0.23	0.48	0.63	1.77	0.48	0.15	0.15	0.60	0.22
February	0.47	0.35	0.78	1.26	1.59	0.31	0.15	0.32	1.35	0.18
March	0.88	0.62	0.95	0.97	1.62	0.49	0.96	0.58	1.23	0.52
April	1.17	1.26	1.23	1.25	1.20	1.03	0.39	0.93	1.02	0.34
May	1.15	1.17	0.98	1.06	0.93	0.98	0.54	0.50	0.94	0.80
June	0.50	1.06	0.48	1.04	0.66	0.54	0.53	0.54	0.66	0.50
July	1.07	1.17	1.00	1.24	0.65	0.46	1.01	1.67	0.66	0.67
August	1.22	0.65	1.20	0.74	1.95	0.68	0.68	2.24	0.92	2.12
September	0.67	0.60	0.68	0.21	0.87	0.80	0.63	1.03	0.60	1.17
October	0.74	0.70	0.88	0.45	1.91	0.29	0.48	0.72	0.26	0.43
November	0.79	0.52	0.70	0.35	0.68	0.14	0.34	0.54	0.39	0.32
December	0.39	0.38	0.86	0.33	0.32	0.13	0.28	1.22	0.20	0.55

TABLE 2.—MONTHLY MAXIMUM PERCENTAGES FOR 1919 TO 1921, INCLUSIVE.

Month.	1919.	1920.	1921.
January	0.37	1.25	0.59
February	0.94	3.32	0.37
March	2.40	2.44	0.94
April	1.58	1.74	0.55
May	0.84	1.48	1.08
June	0.91	1.13	0.89
July	4.82	2.02	1.20
August	3.74	1.78	3.62
September	2.17	....	4.01
October	1.97	0.43	0.91
November	0.87	....	0.57
December	2.35	0.40	1.62

Table 3 gives the volume of soil, in acre-feet, that has been transported in suspension over the deltaic ridge behind the levee system, for each month and each year since 1912, the greater part of this quantity having passed the diversion site. These quantities are for loosely deposited soil and were calculated by using the formula adopted by the office of U. S. Reclamation Service at Yuma, which is:

Acre-feet of silt = acre-feet of discharge × percentage of silt by weight × 0.727

The coefficient, 0.727, is the ratio of weight of 1 cu. ft. of water to that of 1 cu. ft. of loosely deposited silt $= \frac{62.4}{86} = 0.727$.

TABLE 3.—MONTHLY VOLUMES OF SOIL, IN ACRE-FEET, TRANSPORTED IN SUSPENSION TO THE COLORADO RIVER DELTA, FOR 1912 TO 1921, INCLUSIVE.

Month.	1912.	1913.	1914.	1915.	1916.	1917.	1918.	1919.	1920.	1921.
January	477	225	1 298	2 119	32 770	1 581	294	114	2 419	446
February	1 136	587	3 167	13 040	17 184	712	175	633	20 057	319
March	4 438	1 841	5 355	5 599	24 121	1 578	5 572	1 493	8 146	2 396
April	9 618	12 311	10 613	14 637	16 451	10 217	1 399	6 604	7 081	1 531
May	19 875	18 819	21 952	21 295	22 247	19 932	5 835	7 091	17 441	14 205
June	22 765	20 439	21 994	19 802	15 602	20 000	13 075	6 910	35 559	23 095
July	21 117	9 395	21 118	15 147	9 813	18 436	17 565	11 144	11 084	12 350
August	10 910	1 859	10 018	2 650	20 465	6 423	2 098	5 167	5 089	29 609
September	2 124	1 579	1 961	112	3 345	625	479	368	1 030	11 451
October	2 791	2 530	4 398	905	20 313	512	821	624	306	1 204
November	3 424	1 493	2 600	615	2 920	286	848	615	137	679
December	872	894	4 521	584	1 028	282	664	7 084	389	1 871
Yearly totals	99 547	71 972	108 995	96 505	186 259	80 584	48 825	47 847	108 738	99 156

These figures total 948 428 acre-ft. for the 10-year period. Had 70% of this quantity been deposited on the Bee River Ridge behind the levee system, over a meandering width of 4 miles and a length of 25 miles, this area would have been raised more than 10 ft. since 1912.

The transportation of bed sand is not included in the quantities, as the percentages of silt given are a measure of the turbidity. The depth of scour of the river at Yuma and at Andrade, Calif., during periods of high flood, is as

much as 20 ft. lower than the elevation of the normal river bed during low-water periods. Assuming a conservative deepening of 9 ft. for a width of only 600 ft., the volume of soil moved in each 100 miles of the river is 105 600 000 cu. yd. As much of this soil probably falls behind the traveling flood wave to fill the river bed again at locations down stream from its former position, its travel toward its destination on the delta may not be as rapid as that of soil transported in suspension, but undoubtedly great quantities are moved along the river in this manner.

The capacity of the river for erosion in its own alluvium is best conceived by an example. Table 2 shows the silt content to have been more than 4% and, for most of the year, less than 0.5 per cent. If a small part of the flow is diverted through its own alluvial deposit and its velocity is greatly increased, this part has the power to absorb and transport in suspension a higher proportion of silt. Assume that it gathers only 1% additional silt and that the flow is only 2 700 sec-ft. This flow is 100 cu. yd. of water per sec., which is equivalent to 1 cu. yd. of soil transported per second, or 86 400 cu. yd. per day. With such rapid erosion, provided increasing volumes of water are available to the diverted course, any channel can be quickly enlarged so that the entire river flow will soon follow the new course, especially if its old direction on flatter gradients is effectively blocked.

The capacity of the river for soil deposition, or the forming of silt bars, is conversely illustrated. If the flow was into ponded water, or was stilled by dense foliage of thick vegetation, 86 400 cu. yd. of soil would be deposited daily for each 2 700 sec-ft. of inflow charged with a turbidity of only 1 per cent. Confine any such possibility for deposition in a narrow channel, and it will quickly be silted in and become non-existent.

Work Undertaken to Commence the Diversion

The work to be done was based on the following conclusions:

(1) An initial flow of 5 000 to 10 000 sec-ft. of diverted water, having a depth of several feet, and confined to a channel having a slope that would produce erosional velocities in the alluvial soils, was judged to be the minimum necessary to commence erosion and to maintain flow conditions. A shallow depth of flow would be of no service; therefore, the bottom of an excavated channel must be well below the water surface of the river. The water surface of the river was about Elevation 68 for low-water conditions and Elevation 77.4 during the high flood of June, 1921. (See Fig. 4.) The bed of the river was at Elevation 62 for low water and was probably scoured to about Elevation 40 or Elevation 45 during high floods.

(2) The minimum excavation necessary to produce this flow required that the channel have maximum hydraulic radii for depths of 4 to 8 ft. of water. Such a channel would be semi-circular in section, but a rectangular section of the same top width and depth was favored, as undercutting was desired.

(3) As the diverted initial flow is only one-fortieth to one-twentieth of that of high flood, it is apparent that, to make the diversion complete, twenty to thirty times the yardage excavated, must be eroded and transported by the river in following the diverted route.

(4) An excavated width of more than 200 ft. would be necessary for the required initial flow. The dumping of the excavated soil limited the width to about 100 ft.; comparisons of discharges and velocities for one channel, 216 ft. wide, for two parallel channels, each 108 ft. wide, and for three parallel channels, each 72 ft. wide, were made, as shown in Table 4, for each foot-depth of flow.

TABLE 4.*

Width by depth, in feet.	Wet areas, in square feet.	Wet perimeters, in feet.	Hydraulic radii.	Maximum silting velocities, in feet per second.	Velocities for 52.1 ft. to mile, in feet per second.	Discharge, in second-feet.
For Equivalent Single Channel, 216 Ft. Wide.						
216 by 1	216	218	0.98	0.98	1.23	266
216 by 2	432	220	1.96	1.53	2.02	873
216 by 3	648	222	2.92	1.98	2.68	1 737
216 by 4	864	224	3.85	2.38	3.25	2 808
216 by 5	1 080	226	4.78	2.74	3.76	4 060
216 by 6	1 296	228	5.70	3.08	4.23	5 482
216 by 7	1 512	230	6.57	3.40	4.64	7 015
216 by 8	1 728	232	7.45	3.71	5.02	8 675
For Each of Two Channels, 108 Ft. Wide.						
108 by 1	108	110	0.98	0.98	1.20	259
108 by 2	216	112	1.93	1.53	2.00	864
108 by 3	324	114	2.84	1.98	2.64	1 711
108 by 4	432	116	3.72	2.38	3.18	2 748
108 by 5	540	118	4.57	2.74	3.65	3 942
108 by 6	648	120	5.40	3.08	4.08	5 288
108 by 7	756	122	6.20	3.40	4.47	6 759
108 by 8	864	124	6.97	3.71	4.82	8 329
For Each of Three Channels, 72 Ft. Wide.						
72 by 1	72	74	0.97	0.98	1.20	259
72 by 2	144	76	1.89	1.53	1.98	855
72 by 3	216	78	2.77	1.98	2.59	1 678
72 by 4	288	80	3.60	2.38	3.11	2 687
72 by 5	360	82	4.39	2.74	3.56	3 845
72 by 6	432	84	5.14	3.08	3.95	5 119
72 by 7	504	86	5.86	3.40	4.31	6 517
72 by 8	576	88	6.55	3.71	4.61	7 966

* Value of n used is 0.0225.

It was decided to excavate the two 108-ft. channels, in preference to the 72-ft. widths, as there would be less likelihood of the larger channels becoming choked with the large masses of floating driftwood. When dredging was commenced on the first shannel, the bottom width was reduced to 93 ft., with side slopes of $\frac{1}{4}$: 1, in order that the dumping distance in the deepest excavation would not be too great for the 50-ft. boom on the dredges.

(5) As shown in Fig. 4, the original plan contemplated a grade of 2 ft. per mile, leaving the river at Elevation 67, 10 ft. below the banks, and changing to a grade of 4 ft. per mile at Station 112. After excavation had progressed up stream from the arroyo toward Station 112, it was decided to lower the

grade 3 ft. to Elevation 64, at the river. This explains the level grade line shown.

To eliminate this level grade line a further improvement was begun. As the first channel was nearing completion in December, 1921, a cut, 60 ft. wide, was made on the grade of 2 ft. per mile from Elevation 64, thus effectively lowering the original bottom 3 ft. As shown on Fig. 4, this cut was never completed, however, and as it developed later, no change from the original grade of 2 ft. per mile from Elevation 67 was necessary, as there would have been an entry head of 8 ft. when the cut was opened.

It was planned to begin the diversion early in January, when low-water conditions usually prevail and when the river could be controlled. At that time of the year, the flow is sometimes as small as 5 000 sec-ft. and, for many months, it may continue at less than 20 000 sec-ft. Under these conditions, a dam could be built below the inlet of the diversion, and such discharges as were available could be used to begin recession and enlargement of the channel prior to the annual summer flood that has scouring velocities averaging 8 ft. per sec., on its present slope of about 1 ft. per mile.

As the initial capacity of the excavated channel was only a small proportion of the capacity of the river channel during flood, and as the river was on a ridge, no dam could impound the flood flow. Discharge in excess of the capacity of the diversion channel would quickly overflow the river banks and, for this reason, the dam was built only 3 to 4 ft. higher than the banks, and a small levee, of similar height, was built along the north bank from the dam to the Ockerson Levee, in order to confine the overflow to the south bank, up stream from the diversion channel.

The foundations and end abutments of the dam could not be made impervious, because of the nature of the soil. A strong obstruction in the river channel was also not considered desirable until sufficient enlargement had been demonstrated, because, if the levee along the north bank should fail at any point, the river might change its direction to the north and abandon its course into the diversion channels. For high freshets prior to the annual flood, this danger did not exist, as such freshets only lasted a few days, and the topography and dense growth to the north are such that a large stream flow with a falling gauge could not cause erosion.

Because of this element of danger, the dam was made the weak link, its function being to divert the low-river flow so that as much erosion as possible could be obtained by freshets and by the increasing flow of the annual flood. If these quantities happened to increase gradually, the complete diversion might have been made without the breaking of the dam. It would seem, also, that for such ideal conditions there must be a free outlet at the south end of the channels, and the fall of the country makes this appear to be feasible.

Fig. 5 shows the work that was done. Three channels are shown, but only the east channel was made sufficiently deep and long to function effectively prior to the June flood. The other two channels were not intended to be opened to the river until prolonged high water. Fig. 6 is a view of the inlets of the three channels, Channel No. 3 being in the foreground, and Channel No. 1 and Bee River in the background.

The excavation was made with three 1.5-cu. yd. draglines and one 1-cu. yd. dragline. Before the end of January, 657 093 cu. yd. had been excavated by working continuously day and night. Of this yardage, 451 637 cu. yd. was from the main or east channel.

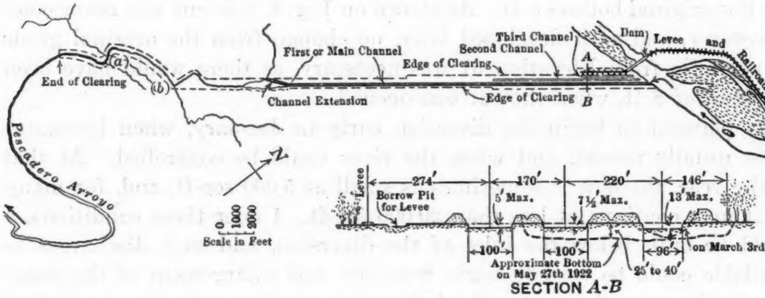

Fig. 5.

Contracts were made with Japanese laborers for clearing the ground along the route of the diversion and for 3½ miles along the arroyo beyond the lower end of the excavated channel. In all, 583.44 acres were cleared south of the river. A cleared strip, comprising 74.52 acres, was made along the north bank of Bee River for the levee and for the railroad extension from the Ockerson Levee. The railroad track was first laid on the surface of the ground, in order to deliver materials and equipment for the dam, as quickly as possible. Pile-driving for the trestle of the dam was commenced on December 1st, 1921, and by December 24th, the trestle had been completed and the track laid across the river.

The Diversion in Progress

It was expected that the channel would be opened to the river, and that the dam would be completed during January, 1922, at which time low flow conditions usually can be expected. However, a flash flood on December 25th, 1921, destroyed the trestle which was then nearing completion. This flood was followed by another on January 4th or 5th, 1922, the water surface in the river reaching Elevation 75.55, less than 2 ft. below the top of the banks. Advantage was taken of this high water, and the barrier, left in the first channel 3 700 ft. from the river, was cut at 7:20 P. M., on January 5th. On the morning of January 6th, the channel was discharging 7 000 to 8 000 sec-ft. at a velocity of about 6 ft. per sec. Fig. 7 is a view of the diversion channel before the barrier was broken on January 5th, 1922. Fig. 8 is a view of the channel, down stream from the inlet on January 10th, 1922. The freshet had passed by January 8th, and the profile (Fig. 9), taken on January 20th, shows the amount that the bottom had been lowered in the first day or two.

From January to May, the discharges in the diversion channel were metered daily at a point 3 700 ft. from Bee River, and gauges, set along the channel and in the river, were also read. These observations shown on Fig. 10, revealed the progress of the diversion.

FIG. 6.—VIEW OF INLETS OF THE THREE CHANNELS.

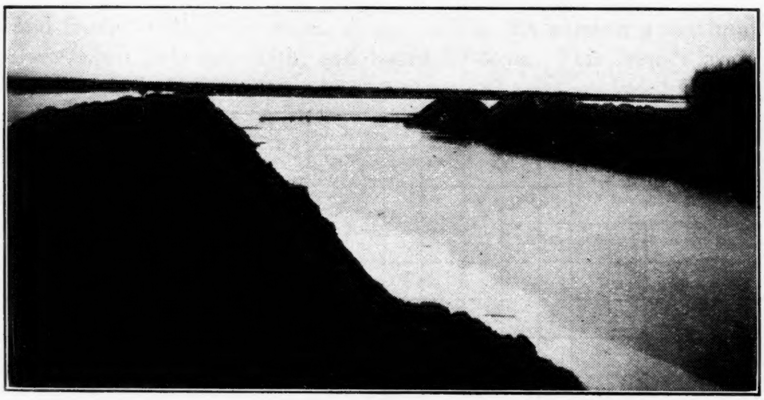

FIG. 7.—DIVERSION CHANNEL BEFORE BARRIER WAS BROKEN, JANUARY 5, 1922. SAND BAG DAM ACROSS INLET.

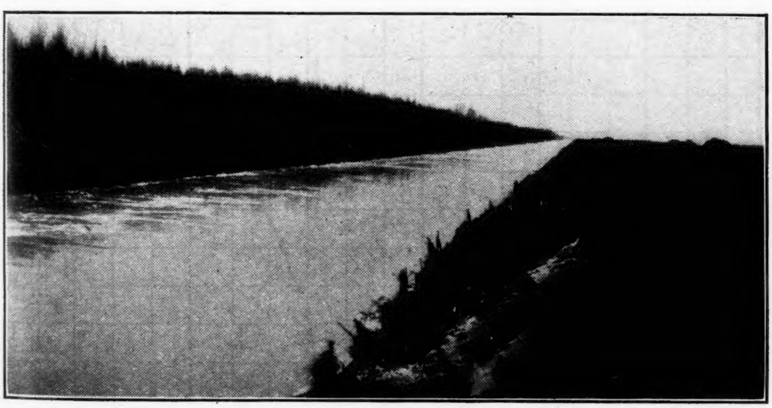

FIG. 8.—DIVERSION CHANNEL DOWN STREAM FROM INLET, JANUARY 10, 1922.

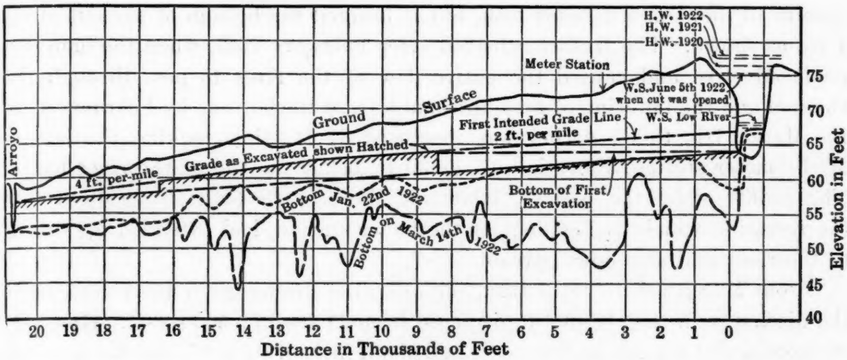

FIG. 9.

From January 15th to February 11th, 1922, the discharge of the river was very low, and only a small quantity of water passed through the diversion. A second freshet with twin peaks, shown on Fig. 10, reached a maximum of 14 800 sec-ft. on February 14th, and lasted 10 days. This freshet made the

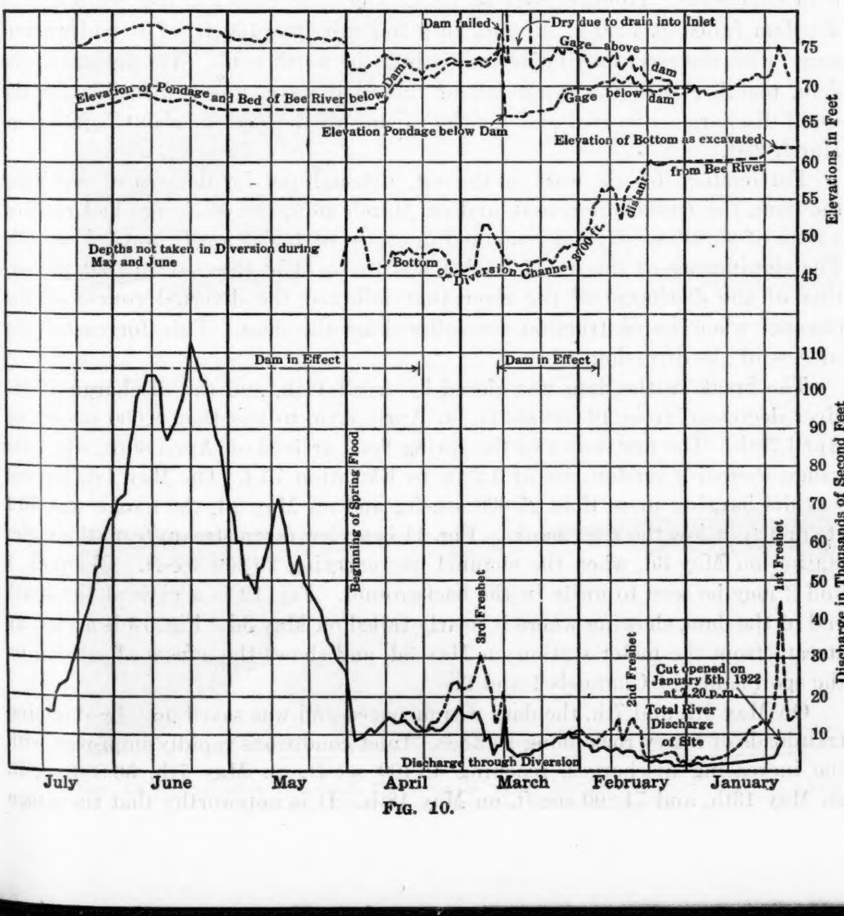

FIG. 10.

repairs to the dam a difficult task, but it lowered the bottom of the cut about 4 ft., as shown. The freshet subsided after February 20th, when the dam was made effective and caused the entire flow of the river to pass through the channel and the lowering of an additional 6 ft. of the bottom, by February 27th. By March 7th, this lowering was increased 3½ ft., thus making the bottom at the meter station, Elevation 47, or 30 ft. below the top of the river banks. The profile along the bottom, taken on March 14th (Fig. 9), showed that the recession which was evident at the meter station, had extended up stream to the river and also down stream.

From March 6th to 18th, 1922, the lessening discharge of the river caused the discharge in the channel to decrease from 11 400 to 5 500 sec-ft. However, the bottom did not silt in, but remained about at Elevation 47, and the pondage up stream from the dam drained into the cut, leaving the dam high and dry.

A third freshet occurred on March 20th, its arrival being very sudden. By 5 A. M., a flood wave of 21 000 sec-ft. had arrived at the diversion, the discharge rising from 6 000 to 21 000 sec-ft., without any warning from up-stream sources. It was thought that the dam would withhold the strain on it, but on account of serious sloughing, a small gap in the middle of the dam was made with explosives. However, before the gap became effective, the north end of the dam failed at 7:00 A. M., and, in a few minutes, 130 ft. of it had washed away, with the river cutting around it into the north bank. The failure of the dam, together with the rapid fall of the flood wave, was responsible for the small discharges diverted and for the raising of the bottom about 5 ft. before March 23d.

Fortunately, for the work on the cut, although not for the cost of repairing the dam, the freshet increased, and on March 26th, the discharge had reached a peak of 30.800 sec-ft. and remained in excess of 15 000 sec-ft. until April 6th. The significance of this third freshet was the evident increase in that proportion of the discharge of the river that followed the diverted course of the channel when no obstruction was offered by the dam. This forecasted the success of the diversion.

The break in the dam was closed by April 11th, and the discharge of the river decreased from 16 000 sec-ft. on April 12th, to less than 9 000 sec-ft., on April 28th. The first water of the spring flood arrived on April 30th, when the gauge showed a sudden rise of 3.2 ft. to Elevation 75.4. On May 1st, the cut was discharging more than 27 000 sec-ft., and on May 2d, the gauge was 76.1 ft., or 1 ft. below the river banks. Fig. 11 is a view down stream from the meter station on May 3d, when the channel was carrying 29 560 sec-ft. Channels 1 and 2 may be seen to unite in the background. Fig. 12 is a view at the south end of the dam, showing where it nearly failed on May 3d. Fig. 13 is a view up stream from the meter station on May 3d, and shows the effects of erosion on the spoil banks of Channels 1 and 2.

On May 6th and 7th, the dam was menaced and was saved only by dumping trainloads of heavy rock along its face. Inlet conditions rapidly improved with the increasing discharges, reaching 42 400 sec-ft. on May 7th, 56 800 sec-ft. on May 13th, and 71 200 sec-ft. on May 18th. It is noteworthy that the gauge

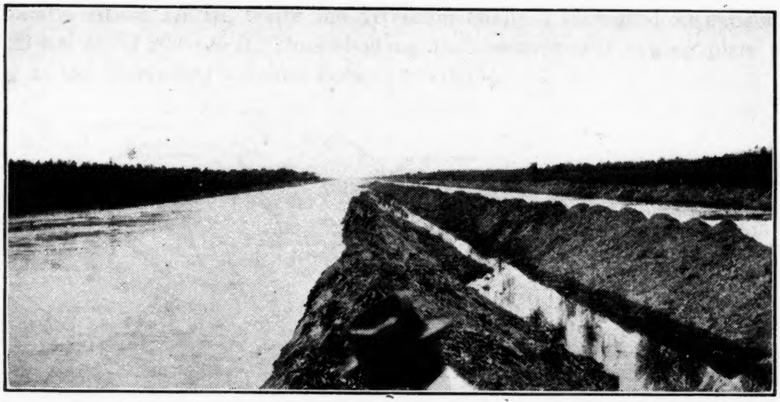

FIG. 11.—DIVERSION CHANNEL DOWN STREAM FROM METER STATION, MAY 3, 1922. CHANNEL CARRYING 29 560 SEC-FT.

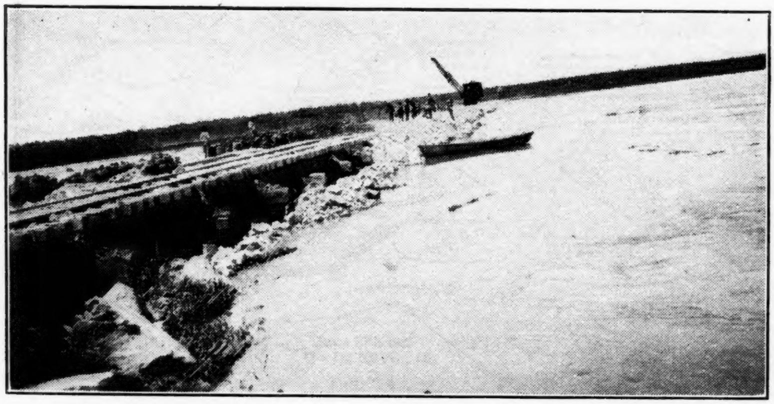

FIG. 12.—SOUTH END OF DAM, SHOWING NEAR FAILURE ON MAY 3, 1922.

FIG. 13.—DIVERSION CHANNEL UP STREAM FROM METER STATION, MAY 3, 1922, SHOWING EFFECTS OF EROSION ON SPOIL BANKS.

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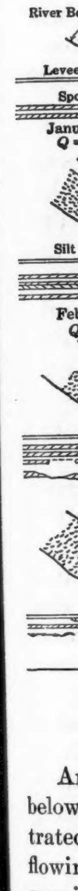

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held steady within 1.1 ft., while the diversion channel increased in capacity from 28 400 to 71 200 sec-ft., thus showing that erosion was taking place as rapidly as the increasing volumes became available.

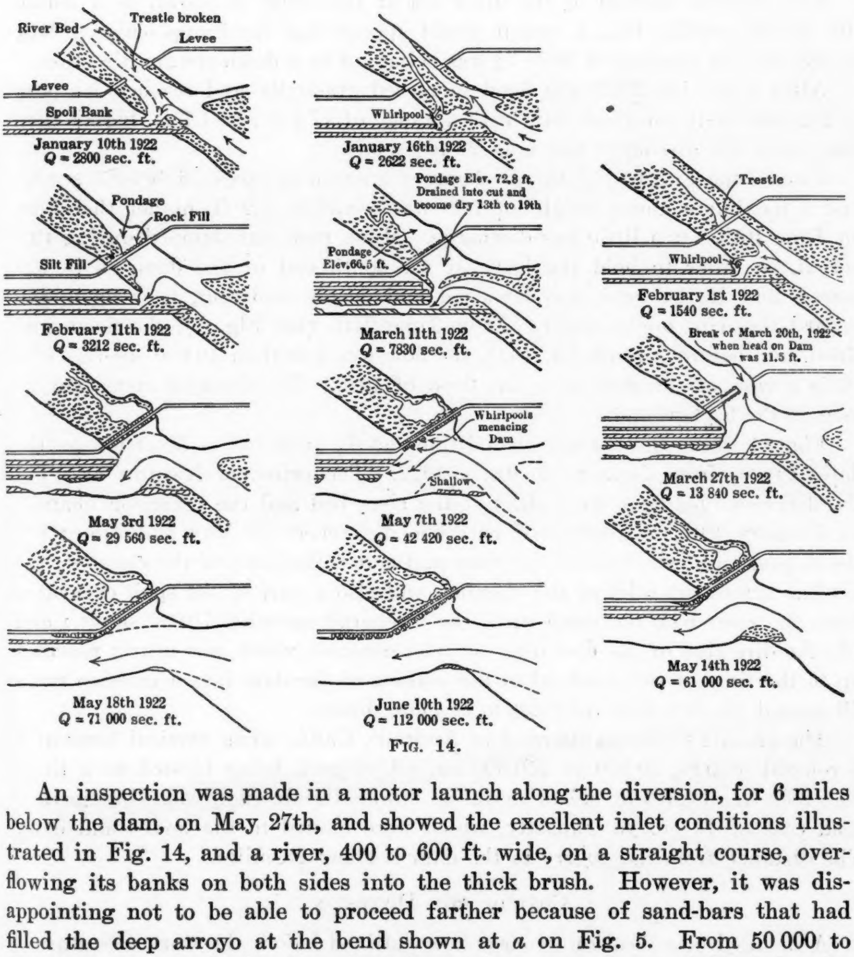

FIG. 14.

An inspection was made in a motor launch along the diversion, for 6 miles below the dam, on May 27th, and showed the excellent inlet conditions illustrated in Fig. 14, and a river, 400 to 600 ft. wide, on a straight course, overflowing its banks on both sides into the thick brush. However, it was disappointing not to be able to proceed farther because of sand-bars that had filled the deep arroyo at the bend shown at *a* on Fig. 5. From 50 000 to 60 000 sec-ft. was flowing into the inlet, and no ground was visible below it; a 3-in. overflow at, say, a velocity of 2 ft. per sec., along the 12 miles of banks (both sides), would account for more than 30 000 sec-ft., and leave a diminishing 20 000 sec-ft. to follow the overflowing arroyo. Fig. 15 is a view of the Pescadero Arroyo in 1921, below the lower end of the diversion cut. There were many beaver dams impounding the water. The kink in the arroyo, the fact that except for a short distance below the point, *a*, no clearing had been done, as well as the curvature of the arroyo to the north, probably caused it to fill with the eroded soils from the diversion. The ground at *a* is about Elevation 59, and the water surface at the inlet was at Elevation 76; therefore, the slope of the water surface was 15 to 17 ft. in 6 miles, or 2.5 to 3

ft. per mile, about three times that of the westerly course of the Bee River; as estimated by the speed of the launch, the velocities were probably 10 to 15 ft. per sec.

The probable bottom of the diversion at this stage is shown as a dotted line on the profile, Fig. 4, and it would appear that the hump between Mile $5\frac{1}{2}$ and the low country at Mile $7\frac{1}{2}$ was being cut in a down-stream direction.

After June 1st, 1922, the flood increased gradually and reached its peak of 112 000 sec-ft. on June 10th, with a gauge of 77.7 ft., or 1.7 ft. higher than that when the discharge was only 28 400 sec-ft.

From June 18th to 23d, the discharge was again in excess of 100 000 sec-ft., and a maximum gauge height of 78.0 was recorded, 0.2 ft. higher than that on June 10th, for a little less discharge. Much rock was dumped during this period, in order to hold the levee at the south end of the dam, as it was feared that if the levee was breached and deeply undercut, back sloughing toward the dam might occur. After June 24th (see Fig. 4), the flood subsided, and before August 1st, 1922, the flow was less than 10 000 sec-ft. Fig. 16 is a view of the dam after the flood of 1922. The diverted river may be seen in the background.

Fig. 14 shows the changing conditions at the inlet to the diversion on the dates given, from January to June, 1922. The principal features are: (1) the difference between the widths of the river bed and the diversion channel on January 10th, as contrasted with the differences in May and June; (2) the formation of silt bars in the river and across the inlet of the channel; the silt bar across the inlet of the channel, and also a part of the spoil excavated from the river bed, remained until the discharge exceeded 70 000 sec-ft.; and (3) the direction of the flow into the inlet channel, which was nearly reversed up to the time of the flood, when the making of the dam into a massive rock-fill caused the flow into the inlet to be more direct.

The granite rock was quarried at Andrade, Calif., along vertical faces of a developed quarry, 50 000 to 250 000 cu. yd. of rock being blasted at a time. The rock was loaded by steam shovel on automatic air-dump side-tipping railroad cars of 16 cu. yd. capacity, which were hauled to the levees and dam. The distance from the quarry to the dam is about 28 miles.

Cost of the Diversion

A detailed classification of costs was outlined before the work was started and Table 5 is taken from the report of dragline operations. In this report, the costs of operation, repairs, and delays, of the dredges can be compared, together with the yardage excavated and the unit costs. These prices include the cost of gasoline and its haulage for $7\frac{1}{2}$ miles.

As much as 1 720 cu. yd. per shift of $7\frac{1}{2}$ hours was attained by these dredges, and 1 100 to 1 500 cu. yd. per machine per shift was common.

An operator's activity chart was plotted daily for the inspection of the dredge crews, and this had much to do with stimulating rivalry for maintaining a high yardage output. A similar chart was kept daily showing the pile-driving operation on the trestle. The trestle cost record is given in Table 6.

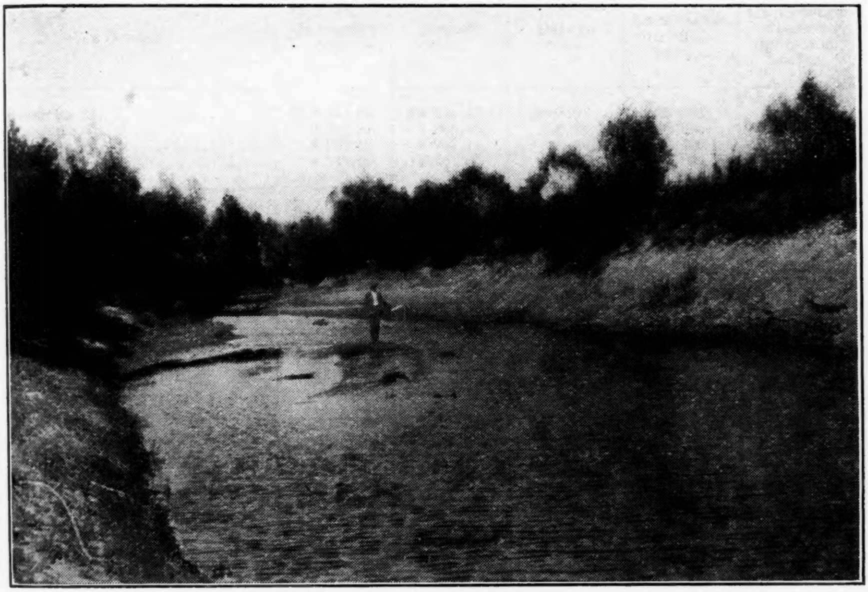

FIG. 15.—PESCADERO ARROYO IN 1921, BELOW LOWER END OF DIVERSION CUT.

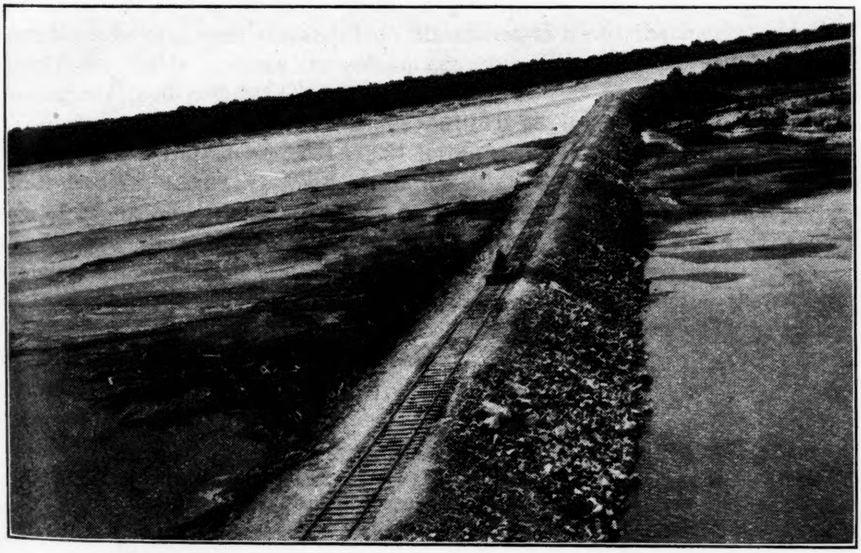

FIG. 16.—VIEW OF DAM AFTER FLOOD OF 1922. DIVERTED RIVER IN BACKGROUND.

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TABLE 5.

Dredge.	Operating cost.	Repair cost.	Delays cost.	Excavation, in cubic yards.	Unit cost of dredge, operation only.
Bucyrus No. 1	$12 004.88	$2 331.40	$883.07	239 984	$0.0500
Bucyrus No. 2	9 431.83	1 866.98	256.22	192 345	0.0490
Bucyrus No. 3	8 088.08	1 747.11	248.89	177 100	0.0456
Monighan	4 648.05	1 431.03	274.05	47 664	0.0975
Total	$34 172.84	$7 376.52	$1 662.23	657 093	$0.0520

TABLE 6.—Trestle Cost Record.

Description.	Number of piles driven.	Total penetration, in feet.	Cost of pile-driving.	Driving cost per pile.	Total cost of trestle built.
First Construction: 1 232-ft. substructure; 837-ft. superstructure	352	8 671	$2 397.12	$6.812	$19 904.91
Second Construction: 700-ft. substructure; 1 080-ft. superstructure	199	5 877	$1 972.46	$9.912	$21 573.28
Third Construction: Extension of north end and wing piling	....	....	....	......	$18 592.42
Total	....	....	....	......	$60 070.61

Tables 7 and 8 show the classification of the entire cost of completing the diversion. Table 7 gives the expenditures to March 15th, 1922, construction operations having been suspended on March 1st to await the increase in the river flow. Table 8 shows the subsequent expenditures to the end of June, making the total cost $413 320.

In the excavation costs (Table 7) $10 110.55 is included for clearing; $3 175.00 for aerial survey; $34 172.84 for operations, repairs, delays, and fuel of all dredges; $26 283.71 for depreciation of dredges; $6 570.93 for depreciation of camps and roads; $17 100.56 for assembling, moving, and dismantling the dredges, camp construction, road, telephones, cableway, metering, tools, supplies, auto-vehicle operations, insurance, overhead, and engineering.

In Table 8 is included the cost of repairing and lengthening the trestle, the strengthening of the dam, and the rock revetment which extends the entire length of the levee along the north bank.

The wages paid were, as follows:

For Dredge Operations:

General foreman of machines	$9.60	per day
General mechanic	210.00	per month
Chief operators, one for each machine	9.00	per day
Operators	1.00	per hour
Engineman	0.70	" "
Groundman	0.375	" "

TABLE 7.—CLASSIFICATION OF COSTS OF DIVERSION TO MARCH 15TH, 1922.

Item.	Quantity.	Unit cost.	Cost.	Total cost.
East Channel, excavation..........	451 637.3 cu. yd.	$16.41	$74 117.94	....
East Channel Extension, excavation....	8 789.6 "	10.95	962.72	....
Deepening East Channel, excavation...	38 091.9 "	16.33	6 223.49	....
Central Channel, excavation..........	121 136.5 "	10.41	12 605.32	....
West Channel, excavation..........	37 437.4 "	9.36	3 504.12	....
(1) Total channel excavation to March 15th, 1922..........	657 092.7 cu. yd.	$14.82	$97 413.59	$97 413.59
Trestle, first construction..........	88 bents, 837-ft. superstructure	....	$19 904.91	....
" second construction..........	51 bents, 1 080-ft. superstructure		21 573.28	....
Dam, built from trestle..........	19 024 cu. yd., rock	$1.05	20 288.94	....
	26 248 cu. yd., silt	0.105	4 042.87	....
(2) Total for damming river to March 15th, 1922..........	....	....	$65 810.00	$65 810.00
Clearing on north bank..........	74.52 acres	....	$2 680.70	....
Levee on north bank of river..........	68 346.3 cu. yd.	$0.30	20 641.89	....
Levee west of channels..........	6 616.3 "	0.51	3 379.32	....
(3) Total for overflow protection to March 15th, 1922..........	74 962.6 cu. yd.	$0.316	$26 701.91	$26 701.91
Railroad, as first laid on ground..........	35 828 lin. ft.	$2.98	$96 843.41	....
Transferring from ground to levee......	25 000 lin. ft.	0.55	13 736.66	....
(4) Total cost of railroad to March 15th, 1922..........	....	....	$110 580.07	$110 580.07
(5) Preliminary cost to February, 1921..........	....	....	$25 180.94	$25 180.94
Total cost to March 15th, 1922..........	....	....	....	$325 686.51

TABLE 8.—ADDITIONAL COSTS, FROM MARCH 15TH TO JUNE 30TH, 1922.

Item.	Quantity.	Unit cost.	Cost	Total cost.
Channel:				
Maintenance..........			$ 866.96	
Purchase of motor-boat..........			1 735.74	$2 602.70
Trestle:				
Repairs and extension at north end, owing to break of March 20th, 1922; also pile wing..........			$18 592.42	
Dam:	29 200 cu. yd. rock.	$1.14	33 451.67	
	3 040 " " silt.	0.10	304.00	
Total for rebuilding and strengthening dam..........	....			$52 348.09
Levee on North Bank:				
Raising..........	23 157 cu. yd. earth.	$0.083	$1 918.90	
Rock reveting levee..........	23 872 " " rock.	0.952	22 737.80	$24 656.70
Railroad:				
Transferring from ground to levee from March 15th to June 30th, 1922..	10 500 lin. ft.	$0.76	$8 026.08	$8 026.08
Total cost of diversion from March 15th to June 30th, 1922..........				$87 633.57

For Trestle Construction:

Pile-driver foreman	$1.00	per hour
Enginemen	0.875	" "
Pile-driver men and helpers	0.75	" "
Bridge carpenters	0.75	" "
Firemen	0.50	" "
Laborers	0.30	" "

For Miscellaneous Work:

Speeder drivers	$144 and $102	per month
Auto-vehicle mechanic	0.70	per hour
Boatmen	0.30	" "
Laborers, first-class	0.30	" "
Laborers, second-class	0.25	" "

Engineering and Clerical:

Instrumentman	$204	per month
Rodmen and chainmen	$132 and 102	" "
Costkeeper	174	" "
Storeman	138	" "
Timekeeper	138	" "

Commissary:

Cooks	$102 and $72	per month
Flunkeys	54	" "

There was deducted from all labor $1 per day for meals furnished at the Commissary.

The Value of the Diversion

In studying the economy of the diversion, the cost of all the work that would have been immediately necessary to the Saiz and the Volcano Lake Levees, had the diversion not been accomplished, must be considered.

A part of this work would have been the construction of a railroad along the Saiz Levee for revetment purposes, all of which would have cost nearly $1 000 000. Also, it would have been necessary to heighten and revet the Volcano Lake Levee, and this would have cost $500 000. To these costs must be added the expected annual cost of repairing and raising these levees and the railroads, for the expected life of the diversion. If the sum of these costs is compared with that of the diversion and its expected maintenance, an idea may be gained of the direct value of the diversion.

Indirect assets will result, because an annually increasing flood menace to the welfare of a great irrigation project has been alleviated for a number of years, or until permanent relief can be assured by the construction of the Boulder Canyon Dam. The diversion tends to promote confidence that will be beneficial in many ways. New capital has already been invested, and crops have been insured against floods in Baja California for the first time.

The period that the river can flow into the Pescadero Country before the flood water again reaches the heights of 1920-21 against the Volcano Lake Levee, can be estimated as follows: The distance along the Bee River channel from the diversion point to Volcano Lake is about 24 miles and the lake is about 36 ft. below the ground at the inlet of the diversion. At Mile 24 on the

profile (Fig. 3), plot Elevation 40 and draw the grade line to Elevation 76 at the inlet, extending the line toward the Gulf. The ground surface along the meanders of the diverted river must become elevated to this line before it will have the same slope that it has toward Volcano Lake.

This line is 15 to 20 ft. above the ground surface beyond Mile 7 and, for a meandering course that will not be less than 40 miles long, 15 to 20 years of deposition would be required to reach a slope similar to that which has been made along the course of the Bee River since 1909. This condition may require the building of a deflecting levee to the south end of Volcano Lake, as the growing elevation up stream from Mile 24 becomes apparent, depending on the direction of the overflow streams.

A conservative estimate of the area below Elevation 40 and above Elevation 20 in Pescadero Basin is 130 000 acres. If it was possible for the river to silt only this area to Elevation 40 (the elevation of Volcano Lake), 1 300 000 acre-ft. of soil would be necessary, and this would require a flow equal to that of the last 14 years, based on the silt deposit for the past decade. It would seem, therefore, that the river could be directed into this basin for at least 15 years, before water to great depth can be stored against the Volcano Lake Levee.

Surveys extending down stream along the diverted river, made as soon as the overflowed country had become dry again, revealed a most interesting unlooked for condition as an aftermath of the flood along the diverted route.

Along the whole length for which the clearing had been taken, the overflowing river has elevated the contiguous ground from less than 1 ft. at the point of diversion, to a gradually increasing height of 13 ft. maximum at 4 to 5 miles down stream. In other words, the river did not scour a deepened channel due to steeper ground slopes than existed on its westerly course, but built an elevated country approximating the gradients on the former westerly course. The new ground surface, elevated by deposition during the ten weeks of the 1922 flood, is shown on Fig. 4; the diagram also shows an intensified slope that now exists between Mile 5 and Mile 10, due to a difference in elevation of about 30 ft.

This newly built country has a maximum elevation along the banks of the diverted river and slopes to nothing at about 3 000 to 4 000 ft. from the river A cross-section would show an excellent demonstration of a débris-cone formation produced by a turbid river.

Conclusion

The Engineering Department of the Imperial Irrigation District, under the direction of F. N. Cronholm, M. Am. Soc. C. E., made the studies and surveys that determined the diversion.

The work during 1922 was conducted by Ray S. Carberry, M. Am. Soc. C. E., Chief Engineer of the Imperial Irrigation District, to whom the writer extends his appreciation for the opportunities given to follow the progress of the diversion, and to place this description of it on record before the Society. F. E. Higley, Superintendent of Construction, is to be commended for his

untiring efforts in maintaining the levees and the dam in an efficient condition during the flood. The diversion might not have been completed during the first flood, if it had not been for Mr. Higley's ceaseless vigilance during the freshets and the 10 weeks' duration of the annual flood. E. E. Kiefer, Chief Cost Accountant, outlined and supervised the collection and classification of costs.

ENGINEERING EDUCATION

THE OBJECTIVE IN ENGINEERING EDUCATION

By Magnus W. Alexander,* Esq.

Technical Papers Presented at the Annual Meeting,
January 17, 1923†

The ultimate aim of education, especially in a democracy, must be the advancement of the moral, intellectual, and physical standard of all the people for the benefit of all the people. The Greek philosopher, Plato, formulated the general concept by stating: "Good education is that which gives to the body and to the soul all the perfections of which they are capable." Modern democracy, however, makes even this formulation broader by emphasizing service to society as the basis of Plato's conception.

The objective of engineering education is to provide society with men who have had that general education so aptly formulated by the Greek philosopher and, in addition, have been trained in a knowledge and understanding of the exact sciences through the applications of which they are enabled to promote the physical well-being of the community. By their habits of exact thinking and constructive outlook, engineers are particularly able to serve as leaders in public and private affairs, because, guided by truths already discovered, they look forward to find safe and sound footings for new structures, rather than look backward at the cold stones of precedent. This pioneer spirit, well grounded and intelligently directed, is the great asset of the engineer.

The pioneer is invariably one who has discovered that his existing circumstances or environment do not permit what he considers the full use of his powers. Whether he is actuated solely by a selfish motive or whether, like Galileo, he is moved by a desire to share with others the fruits of his own thoughts and labors, he finds himself bound by barriers which must be broken down. Strangely enough, these barriers of tradition and inertia are often most strongly supported by those who should be the first to welcome progress. Consider the opposition and hostility to those who sought to give steam navigation to the world. We now stand amazed and amused at the scientists who only a century ago tried to prove mathematically the impossibility of a ship crossing the Atlantic by steam power. Only because men with true engineering spirit, convinced by the results of their painstaking experiments, would not be turned aside from their search for the power which would carry them against adverse winds and currents, can we, to-day, board a palatial steamer with the

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† Continued from March, 1923, *Proceedings*.

conviction that we will be carried to our destination, almost regardless of wind or weather, and will arrive on schedule time.

Let this, as one of many similar examples that could be cited, suffice to sustain the thought that, in our efforts to build a better and truer engineering education, and, for that matter, a truer education of general or specific character, we must develop a spirit of tolerance under which to lessen, even if we may not entirely remove, unreasonable opposition to rational effort for progress.

What is needed to-day in a society that has grown more and more complex in its organization and the relations of which are growing more and more interdependent, is exact, clear thinking on a basis of ascertained fact and experience. It is not enough that the groundwork should appear substantial; it must extend to the bed-rock of truth. How often has quicksand been found in the construction of great railroads or in the erection of lofty buildings? The fact that, in former years, these sands had been considered an insurmountable obstacle to such construction did not daunt the spirit of those who had been taught to search deeper than the shifting sands. By the aid of their science, they froze the sands and made them temporarily useful, while they dug through and beneath them to place solid footings on the unyielding rock.

By establishing the social structure on a solid foundation of truth, and by rearing it with an earnest anticipation of the better things in life, the path of social progress can also be laid out with a fair degree of certainty, and traveled with reasonable safety. The roads to be constructed are many; some have been partly built, others are being surveyed, and others must be projected through what now appear to be inpenetrable jungles. To do so requires knowledge, courage, and character.

The world has waited long for the man with intelligent courage, with skill and vision, who shall pave the way that leads to peaceful co-operation among all men and overcome the discord between nations and the misunderstandings, which, industrially, prevent employers and employees from merging their efforts in the common good. These are the conditions which have obtained in the body politic, despite the efforts to overcome them by men nurtured in older lines of thought. Their removal becomes increasingly imperative.

We cannot question the real desire for peace by those who are to-day striving to settle the economic disruptions that arose out of the World War. We cannot doubt that many employers and employees are earnestly seeking the formula under which both can labor in harmony and satisfaction for their own justifiable ends and the service of society. They all are seeking an adequate solution of the problems; but, do they follow the exact methods of hewing to the line of unprejudiced observation and truth by which the enduring engineering structures have been made possible? How fortunate it would be if the formula of broad social service, instead of the rule of narrow selfishness, were used to find the right way out of present bewildering conditions.

The heart of modern societal organization is in its industries. The foundation of our life to-day is essentially economic, and we are recognizing more and more that the world is economically interdependent. The recent decision, of the British Rubber Growers Association, to limit the rubber produc-

tion in the British colonies in Africa, is not a problem of concern merely to the British rubber industry; it may mean a higher price for rubber in the world's markets, which, in turn, may be reflected in higher production costs of rubber articles in the United States, and may even lead subsequently to a reaction on the demand for these products. Therefore, the employment of the rubber worker in the United States, and the welfare of the storekeepers who depend on him for his trade, are inevitably tied up with this decision.

Similarly, on the disputes in the pastoral industry in Australia, as to the living wage that shall be fixed by the Australian Industrial Court for sheep shearers, hinges the price of Australian wool for world export, the quantity of wool that not only American, but English and other woolen mills, will import, the employment of those engaged in the woolen manufacturing industry, and the ultimate price the consumer may have to pay for woolen clothing. In general, an event in some part of the world may readily bring quick repercussions in other parts thousands of miles away, owing to the highly interdependent character of economic organization. In all actions taken in the economic field, as in other fields, not only, therefore, broad vision and thought based on intelligent selfishness, but also on the common welfare, is necessary, but such actions must be grounded on ascertained facts and their proper understanding and evaluation. Facts constitute the solid groundwork on which we can all build with safety.

The training of the engineer should fit him particularly for leadership in industry as well as in public service, because his method of work brings him constantly in contact with reality, which implies facts and human experiences; and, on the basis of his knowledge of these facts and experiences and of his scientific method of approach, he should be able to formulate workable and effective rules and plans for the solution of the problems that arise.

Just as it is necessary to depend on him who has a correct knowledge of engineering for the erecting of towering buildings and colossal bridges, so the social and economic structure in which we live must be designed and erected by the application of those rules which have their roots in the mathematical formulas of exact thinking, with due consideration, however, for human variables. Lord Kelvin has so truly pointed out to the engineer, as regards work in his professional field: "When you can measure what you are speaking about and express it in numbers, you know something about it, but when you cannot measure it, when you cannot express it in numbers, your knowledge is of meager and unsatisfactory kind." The engineer, however, as a leader in social progress, must broaden this concept of a mathematical character by applying to it due realization and understanding of the fundamentally emotional relationships of persons as well as of groups, bearing in mind that the psychology of a group often differs materially from that of the members composing it.

The particular contribution of the engineer to the world's work lies in the fact that his is a constant spirit of inquiry, of a pioneering endeavor to bring the experience and exact knowledge of the past to apply to definite situations of the present and to probable exigencies of the future; and in the further fact that his is a spirit of open-mindedness born of improvement as inherent in

scientific work. The enhancement of these qualities in the coming engineer is a particular task of the engineering schools; but they must go farther than heretofore in equipping engineers for their work, if it is to be lifted from that of routine to that of constructive leadership. Although it must be the aim to help each engineering student to make his maximum contribution in the common service, the engineering school must cultivate that element in his personality through which he will reach the highest level of usefulness. Without neglecting to teach the necessity for united and correlated action, the instruction must emphasize individuality. It must draw out what is best in each one, searching always for means to promote the good, and eradicating those elements which would oppose true service. The schools have merely arrived at the gateway of character building, and must now discover paths which lead to trees bearing wholesome fruit.

Fostered by necessity (that great mother of invention), engineering schools have developed according to their various and several opportunities. They seized the lamp of science, but, in order that they might pass it on to shine more radiantly and more fully, they had to erect first the material structures in which to live and labor, and then to search and bring together the store of known knowledge and a body of earnest men to vitalize and make it productive of immediate good and further growth. That they have done their work well is shown by their remarkable development. Due to circumstances inherent in the necessary steadfastness of educational institutions, these schools will, from time to time, lag behind in the march of progress, and it is then that we must take stock to see how they may be brought once more into synchronism with their industrial period and with advanced social thought, so as to assume their rightful position in the great social, political, and industrial progression of the nation and the world.

CITY PLANNING

PARKS AND PARKWAYS

By Linn White,* Esq.

Technical Papers Presented at the Annual Meeting, January 19, 1923†

The original definition of a park was "a large piece of ground enclosed by the Monarch's grant, or by prescription." The word later came to be applied to grounds about the homes of the nobility—a gentleman's estate. Thus, the earlier conception of a park implied restraint—exclusiveness. At present, particularly in America, it suggests the opposite—the sense of freedom—a heritage of the people.

In the history of the development of the park and the park idea, the course of freedom can be traced. In Paris, France, the city of parks and parkways (boulevards), the ancient walls around the fortified city were leveled to the ground in 1670 and what are now the principal boulevards were formed on their site. The Bastille was literally torn to pieces in 1789 by the people, and in its stead there is now the Place de la Bastille with its "Column of July" marking the beginning of the French Revolution. So it is throughout Europe, in all the countries where the development of freedom can be traced, the evolution of the park from a closed possession of the few to an open place of recreation for all the people, may be noted.

In the United States, where the rights of the people are paramount, the greatest development of the public park would be expected—and this is the case. American landscape engineers may go to the older countries for inspiration, but they do not go there to learn how the parks should be situated convenient to the people who need them most, or how the lines of transportation are best planned to get to the parks, what recreational facilities are most practical, or how community health, morals, and respect for law are affected by the proper administration of parks.

Public parks may naturally and properly be placed in two general divisions, Natural Parks and City Parks, and these, in turn, may be subdivided according to development and use. As custom and the scarceness of park literature have not established a definite park nomenclature, other names may be found for these divisions. One might say Natural Parks and Artificial Parks, but this would be bad, as all successful parks, even formal public gardens, must follow the bent and teachings of Nature. Natural parks have been called Wild Parks

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† Continued from March, 1923, *Proceedings*.

or Rural Parks. City Parks, also, may be Natural Parks in the sense that the natural conditions and contours may be preserved although the park is situated within a city.

The City Park may be developed and adapted to the highest needs of the community from most unpromising beginnings. Barren sand lots, marshy wastes, shallow waters of lakes and rivers, even forbidding city dumps, may be the ground on which the park builder practices his art, in fact, often must be, because he seldom enters the field until the choice has been made to satisfy industrial and residential needs.

Natural Parks, however, are generally situated where Nature has moulded the landscape and the best that man can do is to preserve what he finds and make it accessible.

The writer will attempt to classify the more obvious sub-divisions of the two general divisions of parks, but this classification is by no means conclusive and, in fact, probably none can be made conclusive, as there are too many common and overlapping characteristics:

Natural Parks:

National Parks, National Monuments, and Battlefield Parks;
State and Interstate Parks; and
Forest Preserves and other Rural Parks.

City Parks:

Large parks within city limits or improved to meet city conditions;
Parkways and Boulevards;
Public Squares and Gardens.
Arboretums.
Zoological Gardens.
Recreation Parks (Park Social Centers).

In this list, the first and last items—National Parks and Recreation Parks—represent two extremes of the development of the park idea, both of which are essentially American.

There are eighteen National Parks, with a total area of 10 739 sq. miles. They extend from Lafayette Park on Mt. Desert Island, off the coast of Maine, to Hot Springs, Ark., and the great Yellowstone Park in Northwestern Wyoming. Rather unfortunately, most of the National Parks are in the West, because most of the suitable territory in the Eastern States had passed into other uses. The first National Park was established in 1832 at Hot Springs, Ark.

At Concord Bridge, the beautiful statue of the Minute Man is standing isolated in restricted and insufficient surroundings. The writer remembers finding along an almost deserted byway in the hills of Massachusetts, with woods on one side and a briar-grown fence on the other, a gray moss-grown stone with crude carving commemorating some tragedy of the earlier Indian wars. Perhaps it is not yet too late to accomplish something better in this historical section of the country.

There are thirty-six National Monuments, such as the Petrified Forest of Arizona and the Muir Woods of California. The Battlefield Parks, such as Gettysburg, Chickamauga, and Vicksburg, are similar in certain characteristics to the National parks, and are to be distinguished mainly by differences in administration.

In some respects, the Battlefield Parks resemble the improved city parks in that a large number of monuments and markers are erected and numerous roads built, but, in other respects, the original character and contour of the ground is preserved. Forest Preserves, such as those at Chicago, Ill., and other Rural or Wild Parks are of a different character and might be classed as City Parks, although situated outside the city limits, except that the general idea is to preserve natural conditions, which seldom can be done to any great extent in truly city parks.

A brief description of the Forest Preserve of Cook County, in which the City of Chicago is situated, should be of interest, as it is one of the largest and most recent enterprises of this kind to be started, although it is by no means yet complete.

The Forest Preserve District of Cook County was organized by an Act of the Illinois Legislature, and was ratified by popular vote of the County in 1914. The Board of County Commissioners administers the affairs of the Forest Preserve, and funds are provided by County bond issues. The lands are acquired by purchase or condemnation, the area now comprising 24 806 acres, stretching in irregular tracts from the southern to the northern boundaries of the County, a distance of a little more than 60 miles.

As the name would imply, the intention is to take principally wooded tracts, although incidentally some open ground is acquired. Southwest of the city lie the Palos Hills which are an irregular group of hills and ravines, a glacial terminal moraine, well wooded, but not especially good land for agriculture. Here, the largest tract of land, 6 688 acres, has been acquired. Most of the other tracts border on streams like the Des Plaines River, and include numerous places of historical interest associated with the early settlement of the country, which are being properly marked and preserved. Other activities of the Forest Preserve are the construction of roads connecting with county highways, re-establishment of old trails, preparation of public golf courses, shelters, preservation of springs, establishment of camp grounds, etc. The Board publishes maps and folders showing motor roads and car lines, reaching different parts of the Preserve, and offers special opportunities for the establishment of summer camps. Thousands of city dwellers have availed themselves of these camping facilities, not only for vacations, but for the entire summer, and the enterprise is still in its infancy.

Some of the larger projects of the Forest Preserve District are the establishment of a great arboretum and a zoological garden which will be one of the greatest in the United States.

At Riverside, Ill., about three miles outside the western limits of the City of Chicago, 300 acres of land have been donated by a public-spirited citizen for this purpose. The Chief Engineer of the Forest Preserve has recently returned

from a trip of inspection of the principal Zoological Gardens of the old world, and active work of construction will begin soon. It will be the intention to give every animal as nearly as possible its natural environments.

The organization of the Chicago Park System doubtless appears quite complicated, as indeed it is, in some respects, but it is, nevertheless, a workable plan.

There are three large park districts in the city, the South, West, and Lincoln (North), which include about four-fifths of the incorporated area of the city. Besides these districts, there are fourteen small districts, some of which maintain only one or two neighborhood parks or playgrounds.

There is also a Bureau of Parks, Playgrounds, and Bathing Beaches, under the direct control of the City Council, which directs and administers sixty-five small parks, four public natatoriums, and four bathing beaches. Most of the parks under the jurisdiction of this Bureau are small, and many of them might be classed as squares, as they differ in size from a few acres to a fraction of an acre.

The number and area of the parks and boulevards controlled by the three large systems are as given in Table 1.

TABLE 1.

	South Parks.		West Parks.		Lincoln Park.	
	Number.	Acres.	Number.	Acres.	Number.	Acres.
Large parks...........	6	1 702.61	4	719.55	1	517.59
Recreation parks......	15	350.68	11	60.15	4	24.36
Other small parks.....	4	67.50	5	39.71	2	8.63
Boulevards............	32.98 miles	448.07	32.5 miles	457.9	11.75 miles	92.05
Bathing beaches.......	3		....	....	2	......
Total area............	....	2 568.86	....	1 275.31	....	642.63

All the Park Districts exercise their functions under certain Acts of the State Legislature, either general or special, and are independent of the Mayor and Council.

The organization of the South Park District by special Act of the Legislature is as follows: Five members of the Board control the affairs of the District under the corporate name of the South Park Commissioners. These Commissioners are appointed by the twenty Circuit Court Judges of Cook County, one commissioner being appointed each year to serve for a term of five years. Thus, it is practically a continuing body and is able to formulate a definite policy with a practical certainty of its being continued from year to year. The jurisdiction of the South Park Commissioners extends over practically all the corporate territory south of the Chicago River, 92.65 sq. miles, which includes the Loop, or central business district, and the south and southwest residence and industrial districts.

The Recreation Park has had its fullest development in the South Park System of Chicago, in fact, the Recreation Park of Chicago is often referred to as the model for the world. The City of Chicago cannot claim the honor of having originated the public playground or the social center, but it remained for that

city to combine these two ideas with others to form a new and unique municipal feature in the Recreation Park.

Playgrounds are often rather barren enclosures with certain stereotyped play apparatus, perhaps a sand court and, sometimes, a wading pool. Social centers generally convey the idea of a somewhat plain, often unattractive building, administered with the uplift idea in the foreground. The Chicago Recreation Park is first of all a small park with trees, lawns, shrubbery, and many of the features of the formal park—balustrades, cement vases, sunken areas, etc. Each park has a substantial building, generally of concrete with tile roof, of rather broad and rambling design, as much unlike a conventional schoolhouse or institutional building as possible. Each building contains an assembly hall, clubrooms, library and reading room, gymnasiums for men and women, with showers, locker rooms, and the usual conveniences of a public building. In some of the larger and more recent buildings there is a lounging room for men, and bowling alleys in the basement; in fact, the arrangements are designed to be as much like a club as possible.

The Public Library maintains a branch in most of the buildings with a trained librarian in charge. Outside the building, but in close connection therewith, is an open-air swimming pool of concrete which, in the later pools, is lined with terra cotta. There are also outdoor gymnasiums for men and women (or boys and girls), with fully equipped playgrounds for children, and as large a ball field as space will permit. All these features are situated in a park setting, often with pools, lagoons, and bridges.

A director is in charge of each building, with athletic directors of both sexes in charge of the social and athletic activities. The buildings are open evenings and Sundays. The assembly halls are, by assignment, used for dances, lectures, educational pictures—anything except for religious and political discussion.

Regular gymnasium classes are conducted, all clean sports are encouraged and taught, all kinds of contests are arranged, leagues formed, and trophies given. Events such as kite tournaments, lantern parades, artificial flower shows, are continually being devised and given a definite place in the season's program. By a cumulative system of marking in all contested events, the crippled boy who can make and fly the most successful kite, or the little girl with a knack of dressing her doll above the mediocre, is taught to feel as much pride and personal responsibility in the standing of the park organization as the athlete who wins the wrestling event. Clean sportsmanship, with the determination to win, is always the slogan. There is even more in wise administration of the Recreation Park than in skillful designing.

The South Park System of Chicago may be said to be entering on a third stage of park building. The first stage was the establishment of the original system of large parks and connecting boulevards; the second was the construction of recreation parks; and the third is the reclamation of the Lake Front of the southern half of the city and the building of a great parkway, or chain of parks, along this shore line, extending from the mouth of the Chicago River southward to the Indiana State line, a distance of nearly 14 miles, from which

should be deducted 2.5 miles where the water-front is occupied by the steel mills.

The part of this work now under way is the 7-mile stretch from Randolph Street, the northern limit of Grant Park, to 57th Street, joining there the shore line of Jackson Park; also 1 mile of lake front from 95th to 102d Streets, south of the steel-mill district.

The project has often been alluded to as the "Outer Boulevard" or "Lake Front Drive." These terms are not correct, as the territory to be reclaimed from the waters of Lake Michigan is more than 3 000 ft. wide in some places. There will be 1 600 acres included in the project between Randolph and 57th Streets, of which 1 200 acres will be land and 400 acres lagoon. Between 95th and 102d Streets, there will be 108 acres of new made land. When these two sections are complete the area of the South Park System will be increased 60 per cent.

From 57th to 79th Street is a third section of the lake shore, part of which is Jackson Park frontage, which is not included in the plans for the immediate future, but is reserved for later consideration.

For many years the tracks of the Illinois Central Railroad occupied the immediate lake shore from Randolph Street to about 50th Street and, by reason of the possession of riparian rights, no other use of the lake shore was possible.

In 1895, after a long period of litigation and negotiation, the limits of the rights of the Railroad Company between Randolph and 12th Streets were fixed by agreement, and the right of the City of Chicago was established to fill an area east of the railroad tracks to a harbor line defined by the Secretary of War. In 1896, the control of this area, then named Grant Park, was vested in the South Park Commissioners.

Filling progressed for several years partly with city wastes and partly with material dredged from the Chicago River and Harbor. In this manner, in a period of about 12 years, 164 acres of land were made for park purposes, filled to elevations 7 to 28 ft. above water level, where the water was formerly 10 to 15 ft. deep.

For various reasons, like many other public improvements, the finishing of Grant Park has been delayed through a period of years, although plans have been made and changed from time to time as new conditions arose. In spite of its somewhat barren condition, many uses have been made of it, from year to year, for military spectacles, camping ground during the World War, aviation field, etc. It is now well under way to completion as a somewhat elaborate formal park suitable for a front-door yard of a great city.

In 1912, another agreement was made between the South Park Commissioners and the Illinois Central Railroad Company, whereby, in exchange for certain definite increase in width of right of way, the Railroad Company conveyed to the Park Commissioners all its somewhat intangible riparian rights from 12th Street to 50th Street. At the same time, the Railroad Company desired a permit from the City authorities to erect a new passenger terminal at 12th Street, and the City authorities desired electrification of the railroad. As all these matters were closely related, the Park Commissioners were unable to realize at once their plans of building a new shore line to Lake Michigan.

Other negotiations followed, political strings had to be pulled, and the whole matter was delayed.

Finally, in 1919, all parties reached an agreement, and an ordinance was passed in the City Council, accepted by the South Park Commissioners and the Illinois Central Railroad Company, providing for the erection of a new passenger station, electrification of all Illinois Central Lines within the city by a certain time, and confirming the agreement between the Railroad Company and Park Commissioners. The assent of the Federal Government was given to the combined project by a permit issued to the South Park Commissioners to fill in the shallow waters of Lake Michigan along the shore from 12th Street to 57th Street, a distance of 6 miles, and to a width of from 1 500 to 3 000 ft. Thus, the people of the south half of Chicago came again into possession of their lake shore which had been given away 75 years earlier to secure the entry of a railroad. The railroad would probably have entered in any case, and the City Council considered it was driving a sharp bargain to allow the new railroad to build along the shifting line of the lake shore to serve as a kind of barrier for the low lands behind.

It will cost $50 000 000 or $60 000 000 to rebuild the shore line as planned, only $50 apiece for every present inhabitant of the South Side, and the Illinois Central Railroad Company will spend another $50 000 000 or $60 000 000 to complete their part of the closely related improvements. The general plan of the South Shore Park improvement is to extend the present shore line 400 to 600 ft. in width, and leave there a water way, 400 to 600 ft. in width, the entire length of the improvement, and build another strip of land on the outer side of the lagoon.

All the shore lines of the newly made land will be formed by bulkheads to retain the fill. As far as planned and constructed to date, the bulkheads are of the so-called "pile pier" type, consisting of two rows of closely driven piling with wales and tie-rods, varied in detail according to conditions, filled between with stone. Details differ in various parts of the work as the finish of the shore line varies. At some places retaining walls will be built on the bulkheads, whereas at others, low concrete docks will be built for boat landings, or beaches will be paved to the water line, or the banks rip-rapped, stony promontories, etc. Where walls, concrete docks, or pavements are supported on the bulkheads, a line of Wakefield sheeting is incorporated in the structure to prevent leakage of the earth or sand filling through the bulkhead.

Untreated piles and timber are used, as the plans require the cutting of all woodwork to the water level, when finished. To retain the fill temporarily, the piling and stone fill are left at an elevation 5 ft. above low water. Large blocks of rough stone, not less than 1 cu. yd. in size, are used to top off the stone fill within the bulkhead. This stone will be removed and re-adjusted to the final grade or used elsewhere as the shore line receives its final finish. The water along the line of the outer bulkhead, permanently exposed to the lake, is 16 to 20 ft. deep. At exposed points and where the bottom of the lake is soft, banks of heavy rip-rap stone are used to reinforce the piling.

The piles used are from 35 to 50 ft. in length, according to the depth of the water and other conditions. Where the depth of water is 16 ft. or more, the

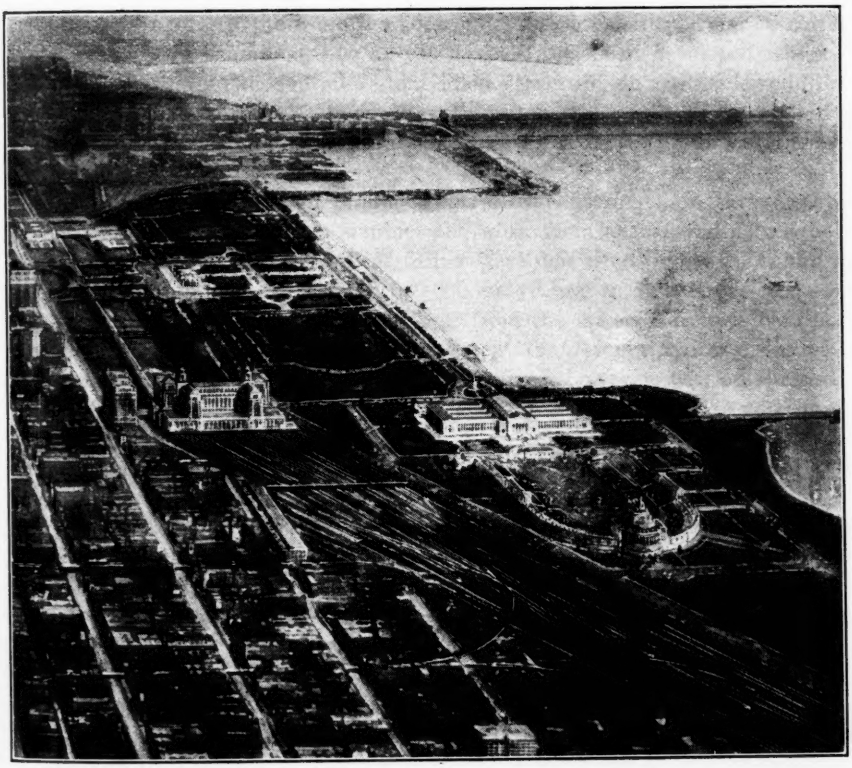

FIG. 1.—AIR VIEW OF GRANT PARK, FIELD MUSEUM, AND STADIUM, CHICAGO, ILL.

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bed of the lake is generally clay which becomes gradually stiffer and harder with depth until hardpan or boulders are found overlying solid rock at a depth of from 70 to 90 ft. Sand lies along the undisturbed shore lines above a 16-ft. depth of water. The conditions differ according to location, some very soft clay streaks being found in places, whereas, in others, particularly between 39th and 50th Streets, reefs of rock extend to near the bed of the lake. It may be necessary to change the type of bulkhead at such places, possibly to cribs or stone mounds. As no construction has been undertaken where rock lies at such shallow depths, and as the investigation of the lake bottom is not complete, no other design of bulkhead has yet been adopted. Many suggested designs for bulkheads involving the use of concrete piles, steel sheeting, concrete caissons, etc., have been examined, but as yet no plan has been developed which can compete with the wooden-pile, stone-filled structure, in cost or flexibility of design for the purpose. Wood beneath water will resist the action of the elements indefinitely, and there are no organisms in the waters of the Great Lakes that will attack wood.

About 20 000 lin. ft. of bulkhead has been constructed in three separated sections, the plans being to conduct the work from as many different points. During 1922, 1 500 000 cu. yd. of filling were deposited by contract, and each year, at the present rate, about 1 000 000 cu. yd. are received from city wastes, miscellaneous excavations, etc. Filling from the latter sources is obtained at no cost to the Park Commissioners, except that of maintaining dumps and dressing to grade. Most of the filling required, however, will be obtained by dredging operations from the bed of the lake. After the filling has been completed, many details remain to be done to produce a finished park territory. A number of farms will be stripped of top soil to cover the new made land to support, trees, shrubbery, and lawns.

Besides the usual functions of a city park, this great Parkway under consideration will serve the people in many unusual ways. The lagoon or enclosed waterway is 6 miles long from Jackson to Grant Parks, 400 ft. wide at the narrowest places, spanned by numerous bridges, and will afford unrivaled opportunities for boating, canoeing, and other aquatic sports; sites will be provided for boat clubs and public boat-houses will be erected. The carrying of passengers to and from the city by barges will be encouraged. What can be more invigorating than to travel to and from business and shopping by such means of transportation, unless it is by motor along one of the two main driveways bordering the open lake where there are no crossings or business traffic? The time by motor car from the Hyde Park residential district to the business center will be cut almost in half when these drives are completed.

There will be at least four large bathing beaches established along this stretch of new lake shore, the first of which, at the southern end in Jackson Park, containing in the beach house 6 000 lockers, is already in use. To insure purity of water, it was determined that all bathing beaches must be entirely open to the lake, without any obstructions or barriers whatever in front.

To build a beach to meet these conditions in 20 ft. of water was a new problem in engineering; in fact, bathing beaches are generally not built. The plan adopted, after much study of natural and accidental conditions along the lake shore, is to construct the main bulkhead along the outer shore line so that,

after the general fill is completed, it may be cut down to 2 or 3 ft. below water for a length of 1 000 or 1 200 ft. to serve as a submerged reef and barrier to retain the sand slope of the artificial beach.

The waves will trip over the submerged barrier, perhaps dig to a depth of 5 or 6 ft. behind it, and expend their remaining energy as on all natural beach slopes. Below water level, the average slope of the sand on Lake Michigan beaches is about 1 on 40. On this basis, the water-line at the proposed beaches should be from 200 to 250 ft. back of the bulkhead line.

None of the proposed beaches has been built according to this plan, but a contract including the first one will be awarded early in 1923. Although there may be unexpected results, careful studies have been made, and it is believed the plan is sufficiently flexible to meet all contingencies.

The time required to complete the entire project as described can scarcely be stated, as so much depends on the financial arrangements. Legislative authority has been obtained for the issuance of bonds to cover the entire estimated cost, but they will be issued from time to time, according to the needs of the work, each issue being submitted to a vote of the Park District. The first issue was for $8 000 000, which will be nearly expended by the end of 1923. No estimate of the time required to complete the project has been made less than fifteen years. Two years have already been actively spent.

The first objective to be attained is to provide a new route from the central south side to the down-town district by a new drive on the new-made land from 23d Street to Grant Park. This involves the construction of a viaduct 634 ft. long between abutments (about 1 600 ft. long, including approaches) and 120 ft. wide, across the tracks of the Illinois Central Railroad at 23d Street; also the widening of South Park Avenue, to 198 ft. from 23d Street to a junction with Grand Boulevard at 35th Street. The cost of the viaduct and approaches is estimated roughly at $1 250 000 and that of widening South Park Avenue, $1 000 000, exclusive of the cost of property to be acquired.

It is expected that the viaduct and widened South Park will be ready for use in the latter part of 1924. By that time the filling and driveway on the new fill will be complete from 23d Street to Grant Park.

Other prominent features connected with the Lake Shore project are the Field Museum at 12th Street, built with funds provided by the will of the late Marshall Field, and the Chicago Stadium now being erected by the South Park Commissioners, just south of the Field Museum. Both these structures are on filled ground. The Field Museum has been open to the public for more than a year. It is built of white marble, and is one of the largest structures of the kind in the world. Fig. 1 is an air view of Grant Park and its surroundings.

The Stadium is of reinforced concrete with artificial stone exterior walls. Its arena is 1 000 ft. long and 300 ft. wide, and the seating capacity when finished will be 60 000.

An interesting feature of the Lake Front project is that this is the first time in the history of park building that plans are especially made for entry of street-car lines into the park. Provision is made to carry all cross-town lines over the Illinois Central Railroad and the lagoon on viaducts.

Car lines have been carried into or through other parks by force of circumstances, but not as a part of deliberately made plans.

CITY PLANNING

ZONING—ITS PROGRESS AND APPLICATION

By Morris Knowles,* M. Am. Soc. C. E.

Synopsis

The object of this paper is to show the engineer the scope and effect of this relatively new method of control of the development of private property. Zoning is really an important part of city planning. Just as city planning is not a single profession, neither is zoning, but rather the result, where best worked out, of the co-operative effort of a number of professions under wise leadership.

Like city planning, however, it is of the greatest importance and interest, particularly to the muncipal engineer. He is concerned with the backbone of city planning, that is, the street layout, thoroughfares, block arrangements, utilities of all kinds, and planning of houses, stores, and factories. Thus he is especially to be active when it is planned to control, regulate, and even fix some of this development by man-made laws.

This paper outlines the rapid growth of zoning, since its introduction in New York City in 1916. Next, the dangers that may result from its evident popularity are mentioned. A definition of zoning and statements of what can and can not be accomplished by it are next presented. The need of the trained legal mind is apparent throughout all steps, both in the preliminary legislation and in the enactment and enforcement of the ordinances.

The work of the Hoover Zoning Committee, in collating present legislation, is stated briefly, together with the outline of the Standard Enabling Act, recommended for the consideration of all Legislatures when contemplating such laws. The need of information on present conditions, in the attempt to plan wisely for the future development of the municipality, leads to a realization that city planning is the main purpose, of which zoning is only a part; zoning is the attempt to render the whole a stable creation.

The types of regulations and subjects covered, although of varying detail, can be classified generally under three main headings: Use of property and structures; heights to which buildings may be built; and areas of lots which may be occupied. Such kinds of drastic legislation need safety valves. One is provided by a Board of Adjustment or Appeal and another by the means for making necessary amendments, with such safeguards as experience has shown wise.

The summary next follows, with the conclusion that the proper aim of zoning is usefulness, and that education and publicity are needed, in order that the demand may be general and result from popular appeal. Finally, the real test is the approval of all the people that it is worth while.

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Progress

New York City Example.—It is peculiarly appropriate that this subject should be discussed in the great city where it originated. The first complete zoning plan in the United States—embodying all the general features as they are now understood—was adopted in New York City in 1916. This was made possible by legislative amendment of the New York Charter in 1914. Additional amendments were made in 1916 and 1917, and zoning was made of general application in the State by later Acts. Prior to this time, under the Acts of various States, only the less complete kinds of zoning were possible.

The New York story, of how zoning has developed and how the interest has grown, is perhaps a most striking example of the influence of the popular will, urgently and effectively presented. Before the passing of the zoning law, there was great agitation because of lack of regulation of congestion, and the encroachment of industry in neighborhoods thought to be desirable for other uses. There was a realization that something must be done. Such interest centered around Fifth Avenue—the brightest business street in New York City—as it was on the way to ruin because of the invasion of work shops for garment and millinery workers, and similar industries. Many such establishments were drawn to this avenue because of the attractiveness of its buildings, nearness to the department stores, for which the goods were being made, and the advantage of a Fifth Avenue address. The result was that crowds of garment workers thronged the Avenue, morning, noon, and evening, crowding out the shoppers and the hotel population.

This resulted in the formation of a protective association, comprised of the owners of the leading department stores, businesses, and hotels. Long before there was a zoning ordinance in New York, there was an edict from the Executive Group of this Association, to the effect that preference in purchases would be made, by the department stores and members of the organization, of goods manufactured outside a given area or zone. This zone extended from Third to Seventh Avenues, and from 33d to 59th Streets. Manufacturers who were within this zone were given eleven months to withdraw, and at the end of that time it was evident that business would not flow to those who stayed. Two full-page notices appeared in the New York daily papers in March and April, 1916. The first was the edict, as follows:

(From *The Evening Post,* New York, March 6, 1916.)

"SHALL WE SAVE NEW YORK?

"A Vital Question to Every One Who Has Pride In This Great City

"Shall we save New York from what? Shall we save it from unnatural and unnecessary crowding, from depopulated sections, from being a city unbeautiful, from high rents, from excessive and illy distributed taxation? We can save it from all of these, so far at least as they are caused by one specified industrial evil—the erection of factories in the residential and famous retail section."

* * * * * *

The second, showing a significantly willing compliance therewith, was as follows:

(From *The New York Times*, Sunday, April 2, 1916.)

"SHALL WE SAVE NEW YORK?

"The co-operative movement to preserve the heart of the City, from 33rd to 59th Streets and from Third to Seventh Avenues, from destruction by factories and to rehabilitate the lower and deserted part of New York City, has met with an unprecedented response from citizens, business firms and other organizations, not only from every part of this city but from all over the United States. The public has put itself squarely behind this great civic movement. One of the most significant evidences of the general acceptance of the plan is illustrated by the unselfish and public-spirited attitude of the cloak and suit manufacturers."

(Then followed a petition by the Cloak, Suit and Skirt Manufacturers' Association, signed by the leading firms of the city, endorsing the movement, and requesting all citizens to support it.)

The effect was even as marked as the result of formal legislation, and the prohibition thus started was soon made effective in ordinance form, after the report of the New York "Heights of Buildings Commissions" and another one on the "Congestion of Population". There is now a new center of clothing and similar industries on the West Side, in convenient, commodious buildings, suitably planned and adapted for these purposes and without detriment to other city facilities. This is a wonderful example of bringing about, by public sentiment first, and later by the same principles being enacted into law, the regulation and control of the use and development of property in the greatest metropolis of the world.

Growth.—Since the enactment of the New York law, zoning legislation has been passed in twenty-seven States and the District of Columbia. In twelve of these States the Enabling Act is comprehensive, and permits zoning of all forms in all municipalities. One hundred and fourteen cities, ranging in population from nearly 6 000 000 to a few thousands, have, to a greater or less extent, established zoning plans. Eighty-one cities have comprehensive ordinances, of which thirty-one relate to use and two to height. In the States where zoning is possible, many more cities are preparing to take advantage of the opportunity thus offered; and in the States where zoning is not possible, preparations are being made for the introduction of the necessary legislation. It would seem, therefore, that experience thus far is demonstrating the value of zoning; that the idea is steadily growing in popularity; is more or less general throughout the country; and the indications are that it will become more so.

A word of caution, however, may be advisable lest its popularity be the means of obstructing its true progress, through well-meaning but too hasty action. Zoning of real value cannot be done hastily and without adequate study and investigation; consideration must be given to local physical con-

ditions, opinions, and usage, and to all the surrounding circumstances. These are important in determining what will be useful—what kind and degree of protection is needed and how it shall be applied. What is good for one city may not be suitable for another, even if the general principles of zoning are the same. Advantage, of course, should be taken of the experience of other cities and the opportunity to review ordinances, but such study must recognize the reasons that have prompted the different methods in different places. A plan for one city cannot be adopted intact for another, and any attempt to do so may prove disastrous and a serious set-back to true progress.

Definition.—Technically, zoning is "The creation by law of districts in which regulations, differing in different districts, prohibit injurious or unsuitable structures and uses of structures and land."

Practical Purpose.—Practically, the aim of a zoning plan is to establish a basis for constructive city growth. All cities have the same general features. They are made up of streets, parks, and other public places, and of land devoted to houses, stores, factories, and similar private uses. Unity in construction, however, is not possible unless there is some measure of control over all land, whether publicly or privately owned; and the only practical method of acquiring such control is through a zoning law.

Such control will promote greater economy, convenience, safety, health, and comfort in industrial, business, and housing conditions. It will guarantee a definite and safe place for industrial investment, will protect home neighborhoods, stimulate home ownership, and assure more contented labor conditions. It will stabilize property values, afford greater security for mortgage loans, and a surer basis for assessment.

What Zoning Does not Do

A.—Zoning does not endanger or retard the progress and general development of any community, large or small.

B.—It does not discontinue or annul private restrictions and deeds.

C.—It is not discriminatory, and does not infringe on constitutional guaranties.

D.—It does not change existing conditions. It is not retro-active and does not prevent the continuance, under present conditions, of existing non-conforming uses or structures.

E.—It does not establish permanent and inflexible provisions that cannot be changed later if necessitated by normal development and growth.

What Zoning Will Accomplish

A.—Zoning encourages prosperous and well-organized community growth.

B.—It makes possible a practical program for future street development and for all public utilities, by determining in advance the future use and requirements of all districts.

C.—It prevents too sudden changes or conversions in the character of districts.

D.—It prevents the intrusion of inappropriate buildings or uses in districts where such buildings and use would injure established improvements.

E.—It stabilizes and protects values and investments, by determining in advance the uses to which property may be put.

F.—It simplifies the problem of traffic, by regulating the height and bulk of buildings, and the consequent street congestion.

G.—It prevents undue congestion of population, by regulations to prevent overcrowding of land.

H.—It insures better conditions for health and sanitation in offices and factories, and minimum requirements for light and air.

Interest of the Engineer

Many interests are concerned in the development of a modern city—the politician, the lawyer, the engineer, the architect, the landscape designer, the business man, the representative of industry and labor, the economist, the student of social problems, the builder, the real estate broker and owner, the banker, the insurance man, and others. Each has his place and his particular point of view in shaping public policy.

It is the function of the engineer, however, to plan the physical features of development: The transit lines, the transportation facilities, the public utilities, the housing and sanitary features. He plans the industrial and business features as well as the physical future of the town. To do this efficiently there must be available a predetermined general scheme or arrangement—one that will remain reasonably permanent. The importance of this is self-evident.

A zoning plan, then, is the basis on which all other plans should be conceived; and it may be said that the zoning plan is primarily an engineering problem. No wonder, then, that the engineer is interested. This is true, however, only in so far as its essential features are concerned; the plan will be of no avail unless established on a firm legal basis; it will be acceptable only if the point of view of all other public and private interests has been considered in its preparation.

Assurance that the plan will stand the test of Court review can be had only through co-operation with attorneys capable of interpreting the meaning of provisions and legal decisions made thereon. Fine distinctions, that escape all but those specially adapted to perceive such refinements, are often made. These distinctions are influenced many times by special circumstances surrounding a particular case, and may have a different interpretation under different conditions.

The zoning plan can be conceived with sufficient breadth of view only through counsel with and co-operation from all interests affected.

Legal Aspects

Zoning is possible under the exercise of the "Police Power" of the State. In a broad sense, the police power of the sovereign authority embraces whatever may be best for the general public welfare of the commonwealth. Another name—an easily understood equivalent—is "Community Power".

Police Power.—No attempt has been made by the Courts to define the limits of the police power—it cannot be done adequately—necessarily, what is for the public advantage may vary according to location, local customs and usage, physical conditions, etc. The influence, therefore, of public opinion on what is for the general welfare is recognized by the Courts. An examination of the various Court decisions will show this to be true, and also that the police power has been extended so as to sanction its application beyond former limits. In the case of the Noble State Bank *v.* Haskell, 219 U. S., 104 (1911), the following statement by the Supreme Court of the United States is made:

"It may be said in a general way that the police power extends to all the great public needs. It may be put forth in aid of what is sanctioned by usage, or held by the prevailing morality or strong and preponderant opinion to be greatly and immediately necessary to the public welfare."

Again, in the case of Eubank *v.* Richmond, 110 Va., 749 (1910), 226 U. S., 137 (1912), the following statement is made:

"That power [the police power] we have defined, as far as it is capable of being defined, by general words, a number of times. It is not susceptible of circumstantial precision. It extends, we have said, not only to regulations which promote the public health, morals and safety, but to those which promote the public convenience or the general prosperity."

In the application of zoning regulations, more restrictive measures in single-family residence districts than in general residence districts (containing apartment houses and similar structures) may not be solely justified on the grounds that the former are more certain to insure public health, safety, welfare, etc., than the latter. It is evident that the measures designed and suitable for the single-family residence districts, if applied to the general residence districts, would interfere seriously with existing development, and would set up such a discrimination, between property already improved and that to be improved, that their wisdom or expediency would be doubtful. On the other hand, more lax regulations, suitable for the general residence districts, if applied to the single-family residence district would be useless, and would permit of great depreciation of property values, that is, "general prosperity".

The application of restrictions, to be justified, must be reasonable, and districts of each type or character, as determined by their development, are treated according to their particular needs. In more general terms, regulations, to be equitable and to be acceptable or to come within the scope of the police power, must affect all similar properties or similar districts throughout the city in a similar manner.

Enabling Act.—As a general rule, cities are not able to establish zoning regulations without some special grant of the police power from the State. Such acts usually prescribe the character and extent of the regulations which may be imposed by the municipality, the limitations of power, the imposition of penalities, and provisions for appeal and amendment.

Some cities have what is called a "Home Rule Charter"; this is particularly true in some of the Western States. Such charters sometimes give the city the authority to zone, by virtue of the grant of charter powers, and special

legislation may not be needed for this particular purpose. However, in such circumstances, it is wise to make sure that the city possesses the police power to regulate the height, bulk, area, and use of buildings and property, and the right to impose different regulations in different districts of a different character.

The Standard Enabling Act, as recommended by the Hoover Committee of the United States Department of Commerce, provides a complete example of the legislation, necessary to delegate police power to the municipality, in order to regulate the uses and development of property and provide all the machinery for doing so.

Section 1 recites the grant of power, with definite limitations and expression of purpose. Section 2 permits the division of the municipality into districts, providing that regulations throughout each district shall be uniform, but the regulations in one district may differ from those in others. Section 3 expresses the general purpose of such legislation, and makes clear the establishment of the "atmosphere" under which zoning is to be done.

Section 4 provides for the manner of the creation of the districts and, together with Section 5, the method of making changes and amendments, and the provision that certain percentages of interested persons may necessitate a vote of three-quarters of the municipal legislative body in order to amend. Section 6 provides for the creation of a special body, to be known as a Zoning Commission (or its equivalent), working as a City Planning Commission, for the purpose of making the original study, holding hearings, and presenting a report to the municipal legislative body on the subject of a zoning ordinance.

Section 7 provides for a Board of Adjustment or Appeal, its membership, rules of procedure, machinery of appeal, and further appeal to the Courts. It is provided that the Board of Appeals shall review any act of the administrative officer on alleged error. There are also certain listed exceptions from the terms of the ordinance which such board may permit.

The board also has the power to vary the terms of the ordinance, where, owing to special conditions, the literal enforcement will result in unnecessary hardship, but, at the same time, the spirit of the ordinance shall be observed and substantial justice done.

Section 8 provides the remedies and penalties for violation of the ordinance. Section 9 states that, where there is conflict with other laws, the higher standards and restrictions shall govern.

Ordinance.—The regulations of a city are established and applied through the usual ordinance or legislative machinery, drawn in accordance with the provisions of the State Enabling Act. Such an ordinance usually consists of one or more maps, dividing the city into different kinds of districts, and statements of the regulations to be applied in each district regarding use, height, and area.

Legal Advice Needed.—The success of a zoning plan depends on the adequacy of the Enabling Act, and the provisions of the ordinance must be such as to come within the limits of the Act and within the scope of the police power. There is need, therefore, of the lawyer, who is familiar with and can rightfully interpret the decisions of the Court and the probable attitude in

specific instances. Some features of zoning are not yet approved by the Courts so widely as to be generally recognized as coming within the scope of the police power. The greatest stress, therefore, should be laid on the importance of having sound, experienced legal advice in the drafting of the ordinance.

Preparation of the Zoning Plan

Planning Commission.—There should be a commission to prepare the zoning plan and ordinance for recommendation to the City Council. This may be the City Planning Commission, if there is one, or a special Zoning Comsion appointed for this purpose. The majority of the members of such a commission should be, in most cases, citizens serving without pay. They should not be hampered by political considerations. The city administration, however, should be represented, and this usually is accomplished by certain of the officials acting in an *ex officio* capacity. Actions of immediate expediency are thus to be given due consideration, as well as the effect on the far-reaching future.

The creation of such commission, the membership, method of appointment, and powers are usually provided through the statute or charter provision. The commission should have more than advisory powers; usually, it is provided that, before any other city authority acts finally on an amendment to any clause of the zoning ordinance, it must be referred to the commission for consideration, and a certain time allowed for a report.

Working Staff.—As the members of such a commission cannot attend personally to the collection of data, the preparation of maps, and the working out of details in the development of the zoning plans, they should be provided with an adequate staff of trained men under the direction of a competent executive.

In addition, they should have the benefit of expert experienced advice. Zoning requires the services of more than a single profession. It requires knowledge of all those things which go to make up municipal development—the proper planning of streets, transportation facilities, public utilities, housing, etc. The services of the all-around civist are needed; but here a word of caution may not be amiss, especially if the commission or staff is inclined to be lazy or lacks diligence. There is real danger, if the expert, however well informed, attempts to foist on a community, without the detailed study of local needs and application of principles derived therefrom, a plan that has worked well elsewhere. The ordinance must also appeal to the local sentiment, and it cannot do so without being a result of local education and development.

Reliable Data Required.—Complete and accurate data are needed, and an intensive study of local conditions and problems is required. Such data will refer to the use and height of buildings, the vacant spaces about them, the character of industries and establishments, the population and densities thereof, the values of land, the location of business enterprises, the tendencies of development, etc. The data should be presented, preferably, in graphical form or on maps, so that they may be comprehended and analyzed readily.

General Procedure.—The commission, after completing the preliminary plans and tentative form of the zoning ordinance, should hold meetings and public hearings, prior to submitting the completed work to the Council. The

Council, after receiving the ordinance, may hold other meetings, and, if it is deemed necessary, may refer the ordinance back to the commission for amendment, or may enact it into law.

It is not possible to emphasize unduly the need of a thorough public discussion of the zoning plan. To be acceptable, the ordinance must meet the approval of substantially the entire community, and must demonstrate its reasonableness to property owners. If the general public is to be able to understand and consider the merits of the plan seriously, it must know why certain things have been done and the reasons for certain regulations. This can be accomplished best through frequent meetings during the work, particularly before small groups where peculiar local problems may be discussed, and where the protection offered to residence property may be explained. Similar meetings should be held for discussion with the owners of business property, with manufacturing interests, real estate associations, and the various civic bodies of the community. The influence of the press will be of great assistance, and frequent articles should be published, explaining the purpose and the effect of the various regulations.

ZONING REGULATIONS

General.—A complete zoning plan utilizes three general methods of regulation: The use of property and structures; the height to which buildings may be built; and the areas that must be left vacant.

Use Regulations.—The purpose of the regulations of the first class is to segregate the various uses of public and private property and restrict such uses to suitable locations. Some land will be found to be especially fitted for certain purposes, industries will naturally be placed near railroads, or near the waterfront, residence districts of the better class will develop remote from industry, and not too near the business section, workingmen's houses will be built near their work, or near adequate transit lines. The types of districts in a city are usually well-defined, and are more or less fixed by existing development, topography, and cleavages, like watercourses or hills.

Zoning should anticipate the continuance of established uses of property, when these are not in conflict with wise development, even if the ideal arrangement might possibly be better. The city cannot be rebuilt overnight, and it is possible only to approach the ideal. Advantage must be taken of the mistakes of the past, but the harm that may already have been done by indiscriminate building is of small account if the future of the city can be protected.

Restrictions as to use are the most important provisions of the ordinance and those most readily understood. In general, there should be regulations for heavy industry, light industry, business or commercial districts, and residential purposes. Some communities will deem it necessary to divide industry into three groups, or to have a secondary commercial district for light manufacturing, and some will have three classes of residence districts. In the speaker's opinion, such minutiæ are undesirable, and are impracticable of exact definition, at least at the inception of the ordinance. Such detail, if utilized at all, should come after the workings of the provisions of the ordinance have been tested.

In some cities, there are no limitations on the use of structures or property in the heavy industrial districts, but usually certain manufacturing processes, which create nuisance or which are dangerous, are prohibited. Dwellings in such districts should not be permitted, except under definite limitations, such as provisions for the use of watchmen and their families, employed on the premises of an industrial establishment.

The light industrial districts are intended for factories which are not objectionable because of odor, dust, smoke, gas, vibration, or noise. In addition, business establishments of any kind are permitted, and residences.

The business or commercial districts are intended for warehouses, wholesale and retail stores, office buildings, hotels, and similar structures. No manufacturing is permitted, other than that required for products to be sold on the premises to the consumer. Other establishments, which would be objectionable, such as blacksmith shops, carpet cleaning works, contractors' plants, public garages (except with special permits), etc., may also be prohibited.

Residence districts should allow single- and multiple-family dwellings, churches, educational and charitable institutions, greenhouses, gardens, and the like, including their usual accessories. Public utilities, by special permit, when found to be necessary (such as telephone central exchange buildings, electric sub-stations without rotating machinery, and gas-regulating stations) may be permitted in the least restricted districts and in some cases excluded. The accessory uses are so varied and so numerous that it is difficult to recount them. It is safer to refer to the classification as "those accessory uses which are customarily incident to the uses permitted in the district, and not involving the conduct of a business."

The conditions under which small garages and stables and community garages are to be permitted should be specified carefully. It is possible, also, to curtail the erection of bill boards, signs, and spite fences by zoning regulations. Although bill boards may be proper in certain places, the Courts have recognized that they should not be placed indiscriminately among houses, schools, and churches. Spite fences may be controlled by limiting the height of fences.

Many zoning ordinances have provided two or more classifications for residence districts, differing only as to the restrictive regulations. Primarily, the purpose is to provide for single-family residence districts. This may be requested by large numbers of people, and may be desirable, but, legally, it may be hazardous unless the power is expressly stated in the enabling act. It may be difficult, in many cases, to justify such procedure on the basis of health and safety. However, the definite limitations of certain districts to the single-family house is becoming more common in zoning ordinances, and the generally favorable attitude of the Courts has been reflected in recent decisions. It has been decided that the police power can be used for the purpose of establishing private residence districts, and it is only a step farther to the classification of such districts.

Exceptions to the regulations pertaining to use, in district classifications, may be provided in the ordinance. Public and community garages, for

instance, are sometimes desirable in residence districts, and can be situated so that they will not be detrimental to adjoining property. Exceptions of this kind are usually made contingent on the desires of the majority of the owners of property in the immediate vicinity. Provisions may be made also concerning uses which do not conform to the standards of the districts in which they are situated. Such uses may continue until they are discontinued voluntarily, or by fire or other destruction, or are altered structurally, or to a certain large percentage of value.

Height Classifications.—Regulations of the height of buildings, particularly in the business sections of a city, are likely to meet with the most active opposition. There is reason for this opposition, and careful consideration and study are required if an equitable balance is to be established between what is theoretically desirable and what is practicable of accomplishment. Office buildings, hotels, and similar structures must be built to a certain height if they are to pay adequate returns on the investment. Such buildings, however, may readily be built too high for economy.

It is well known that many of the lower stories of the high buildings in the canyon-like streets of New York are useless for practically all purposes except storage. In areas where high buildings are crowded together, most of the rooms on the lower floors are inadequately lighted and insufficiently ventilated, traffic congestion is inevitable, public utilities are periodically overtaxed, and both private and public financial loss is the result. There is a definite and real need, which the engineer is qualified to fulfill, to obtain and present carefully digested facts on the existing conditions of high and moderately high buildings; as to light, air, comfort, waste, elevator and corridor space, street congestion, burden on public utility facilities, etc.

The report of the "Height of Buildings Commission",* discloses surprising facts regarding the average height of buildings in lower New York City:

"The high building problem is at present confined chiefly to a comparatively small portion of the lower half of the Island of Manhattan. The average building height in the Borough of Manhattan is 4.8 stories. Ninety per cent. of the buildings do not exceed a height of six stories. The buildings over ten stories in height constitute only a little over one per cent. of the total. Out of a total 92 749 buildings, there are but 1 048 buildings over ten stories in height; ninety buildings over seventeen stories; fifty-one buildings over twenty stories; and only nine buildings over thirty stories."

The average height of buildings in the down-town business section of Pittsburgh, Pa., where the higher office buildings, the larger department stores and the prominent hotels are situated, is only a little more than three stories. in a paper by Mr. H. J. Burton, of Minneapolis,† it is stated that:

"The corner of Wall Street and Broadway was considered the most valuable land in America. It was about 40 ft. on Wall by about 40 ft. on Broadway, and originally had thereon about a six-story building, which earned a moderate profit, after paying the taxes on the assessed valuation.

"The owner of the L-shaped land, in the rear from Wall Street round to Broadway, erected a tall building thereon, using the light and air over the low building on the corner. He had $1 000 000 money and borrowed $1 600 000

* New York, December 23, 1913.

† *Proceedings,* National Association of Building Owners and Managers, 1918.

more to pay for his new building, and earned a moderate net profit from his rentals, after paying his taxes and interest on mortgage. So the man on the corner concluded to tear down his low corner building and erect a tall building, to help pay his increasing taxation. This building took away part of the light and air from the rear building, which presently lost most of its tenants to other new buildings. The $1 600 000 mortgage was subsequently foreclosed, the mortgagor losing his title to the land and his additional $1 000 000 investment in the building.

"The man on the corner did not receive as much net income (after amortizing the cost of his new building) as he originally received on his little low building."

In this same paper it is stated that:

"From 1913 to 1917, the assessed land values, exclusive of improvements, on Manhattan Island south of Fortieth Street, declined more than $186 000 000."

The statement of a New York authority on the subject* is quoted as follows:

"In my opinion after ten years' service, if there were no buildings between Forty-second Street and Broadway, of more than 8 stories, the land value would be more than it is to-day."

The height regulations are not of so great importance in other than the more congested business sections, where land values are excessive. The usual limit in the residence district is equivalent to a height of 2½ or 3 stories. The intermediate districts allow the buildings to be erected to heights of 4, 6 and 8 stories. All these limitations may be expressed in terms of multiples of widths of streets, number of stories, or in some combination of these, but it is rare that all are used in the same restriction. The maximum restriction varies from 125 to 150 or 175 ft., depending somewhat on the size of the city, with higher maxima with set-backs in New York City. Here, again, as in the use districts, exceptions may be made. They apply, in some cases, to institutions, hotels, public buildings, towers, spires, etc. Usually, also, additions to the height of buildings above the prescribed limits are allowed if set-backs from the street lines (in some cases property lines, also) are observed. There is great need of data on the effect of height on the return from the investment in buildings, and the effect on the development of ample street areas and adequate public utility facilities. One is a private, and the other a public, effect, but reflected in a private way by taxation. This is indeed a fertile field for the engineer, for there is much opinion not founded on basic truth.

Area Regulations.—Area regulations may be applied in a number of ways, such as front-yard requirements; rear-yard depths; width of side-yards, inner and outer courts; percentage of lot occupied, and allowable density of population. Such regulations at best are complicated, and all methods are seldom used in one ordinance. Requirements designed to control density of population directly are the most interesting, and may be expressed as the number of families allowed per acre or the number of square feet to be provided for each family.

* *Proceedings,* National Association of Building Owners and Managers, 1920.

Provision also should be made in the ordinance, if possible and legal, for continuing in effect the "set-backs" from the street, where already established by private restrictions or otherwise. This may be done by prescribing the depth of front yards, or the maintenance of a building line, as already established by a large percentage of the structures now built.

In connection with the area regulations, certain exceptions will be necessary, so as not to prevent the erection of a one-family house on a lot that may be too small to provide the required area. It is also necessary to differentiate between inside lots and corner lots, to provide special regulations for through lots, etc.

Administrative Agency.—Zoning ordinances, as a rule, are enforced by the same agency that has to do with building codes, and other forms of building regulations. It is desirable to centralize the control of building regulations, so that permits for all features of construction may be obtained at one time by visiting one office. One of the serious difficulties and causes of annoyance in the building industry is the multitude of offices which must be visited to secure the many needed permits before construction can commence. Anything which can be done to simplify this and to lessen the expense will receive approbation.

Amendments and Changes.—The fundamental purpose of a zoning law is to prevent too rapid changes, which result in the deterioration and modification of property values. Nevertheless, conditions will change, and different regulations will be required. Sometimes the people in a district believe the time has come for a change in use or classification. It may develop that, because of expansion of the community, or from some other reason, industry or business is best fitted for a certain residence district and that land will sell at a higher figure for such purposes than for any other. Then there will develop a popular demand for revision of the classification, and the same authority that has adopted the ordinance should have the power to amend the regulations.

Proposed changes should be studied first and reported on by the Planning Commission. A public hearing should be held, to consider the requests for and the protests against amendments. This should be after the prescribed publication of notice, and hearings and the report of the Commission to Council. If then the request for change is still opposed by a certain proportion of the property owners affected, more than a majority vote of Council should be required for approval of the amendment. This safeguard is wise, and is designed to prevent changes and amendments being easily made because of sporadic, whimsical, and changeable desires in particular cases of supposed and yet limited benefits to be gained by such amendments.

Board of Adjustment.—Because a zoning ordinance is so comprehensive, it is evident that hardship and serious discomfort might be caused by a strict and literal, but unreasonable, interpretation of its provisions, as they cover so many different things in a different way. It is desirable, therefore, to provide a Board of Adjustment or Appeal to which the decisions of the administrative agencies may be appealed and reviewed. Every decision of the Board of Appeals should be reviewable by the Courts on a writ of *certiorari*.

The legislative act granting to the city the right to zone should also make provisions for the creation of such a board of appeals, otherwise the City Council cannot endow such a board with powers to decide certain cases or to make variations in the provisions of the ordinance in order to carry out the spirit of the law and prevent unnecessary hardship. The province of the Board of Adjustment should be definitely and carefully limited to those things prescribed in the act; otherwise there may be gross abuse of the power thus delegated. That there is danger, however, should not deter one from providing such a board or safety appliance; for otherwise the ordinance may fail in its purpose by being declared unnecessarily harsh and confiscatory whenever it is brought before the Courts on a particular and exceptional case.

An example of reasonable variation is that the use of premises may be permitted for the manufacture of building material in a territory where it is evident that this is a convenient and economical way to prepare local materials for the erection of buildings within the district. There may be border line cases, also, regarding public service, enlargement of non-conforming use, continuance after fire or disaster, etc., which can best be handled by such Board of Adjustment.

There may be other types of exceptional cases which are difficult of interpretation by the administrative officer. Provision should be made for these and all others, where there are practical difficulties or unnecessary hardships in complying with the strict provisions of the ordinance. In such cases the Board should vary and adjust, in harmony with the general purposes and intent of the ordinance, so that health, safety, and general welfare may be secured, and substantial justice done.

The following is quoted from the brief of Edward M. Bassett, Esq., as *amicus curiæ* in the Sheldon-Astor New York case:

"In the more complex field of regulation of all buildings and their uses, it would be dangerous if the State legislature did not provide some method of adjustment in exceptional applications for permits. The Board of Appeals is the safety valve in the zoning plan. Its function is analogous to that of a regulatory commission for public utilities, or the pure food administration under the federal statute. The legislature establishes a rule within which the administrative board or commission shall regulate. Rates shall be *reasonable*. Service shall be *adequate*. Appliances shall be *safe*."

In this connection, it is of interest to quote from the decision of the Court of Appeals of the State of New York in this same case:

"The courts below held that no zoning regulation adopted by the board of estimate and apportionment may in effect be repealed or set at naught by the board of appeals by action taken in the guise of a variance. Such conclusion is not only in conflict with the expressed intention of the zoning statutes but likewise the power delegated to the board of appeals directly by the legislature and by the board of estimate and apportionment pursuant to legislative authority."

* * * * * *

"The authority conferred by section 7 upon the board of appeals was to 'determine and vary' the use district regulations, by section 20 to 'vary' any

provision of the resolution in a specific case, where substantial justice will result therefrom."

* * * * * *

"If as the result of investigation the board of appeals shall 'determine', as it did in this specific case, that there existed unnecessary hardships in the way of carrying out the strict letter of the provisions of the zone resolution and that substantial justice would be promoted both to the property owners and the public interest, its power then was enlarged to 'vary', *i. e.*, to modify or alter in form or substance the application of the regulations of the board of estimate and apportionment in the specific case, which it did by permitting the building which the owner proposed to erect upon his lots embraced in the business district to be extended so as to cover the lots fronting on Madison Avenue."

* * * * * *

"Concluding as we do that the legislation under consideration authorized the board of estimate and apportionment to confer upon the board of appeals the authority exercised by the latter board in the instant case, and that the amendments to the regulations, a part of the zone resolution adopted by the board of estimate and apportionment were in consonance with the legislative intention, the orders of the Special Term and Appellate Division should be reversed and the writ of *certiorari* dismissed, with costs to appellants in all courts."

Summary

Practical zoning must be established on a firm legal basis. A proper grant of the police power of the State to the city is pre-requisite. The preparation of the ordinance so as to be safely within the scope of the police power thus conferred is equally important. Therefore, the advice of a lawyer, fully informed on the various decisions of Court cases, is needed.

In the preparation of the plan and ordinance, sufficient study of local conditions is essential; there is danger in too hasty action. The plans and experiences of other cities should be studied, but with a knowledge of why certain methods were adopted. No direct copying is permissible.

The aim of zoning is usefulness; hence, in order to succeed, the plan must be practicable, and there must be a nice balance between what is theoretically desirable and what is practicable of accomplishment. Although many things are desirable, ideals can only be approached. The city cannot be re-made overnight. It is possible to do the best only with what is available, and look to the future for improvement.

The public must be taken into the confidence of those promoting the zoning plan. To succeed, the ordinance must be what the majority of the people want, not the desires of a few. Publicity cannot be overdone. More often than not, there is a lack of understanding in the community on the necessity or desirability for zoning regulations.

The ordinance, in general, should prescribe regulations as to the use of land and structures, the height to which buildings may be built, and the area that must be left vacant on each lot. The districts in which the different regulations are to be applied may or may not be coterminous, or they may be combined. That arrangement which will secure the greatest simplicity should be adopted, and this will depend largely on local conditions.

Provision should be made for amendment of the ordinance, under adequate safeguard, and a Board of Adjustment should be created. The function of such a board is to review any contested decision of the administrative official, to decide border lines and exceptional cases, and to adjust the regulations in harmony with their spirit in cases where a literal but too strict intepretation would cause unnecessary and excessive hardship.

Above all, the plan, to succeed, must contain only those things which are practical of accomplishment. Under the scope of police power, the law recognizes the necessity only of those things which are beneficial to the whole. Esthetic and similar considerations are not true uses of the police power as a main basis. There must be a fair relationship between the public good to be secured and the private injury suffered. Local physical conditions, usage, opinion, and all the surrounding circumstances are important in determining what will be useful, and whether the ordinance will be upheld by the Courts.

The real test of a zoning plan will be: Whether or not it works?

ENGINEERING EDUCATION

Discussion*

By Messrs. Robert Fletcher, F. B. Sanborn, J. K. Finch, George F. Swain, C. M. Spofford, Dugald C. Jackson, Theodore T. McCrosky, T. Chalkley Hatton, J. C. Ralston, Milo S. Ketchum, W. E. Wickenden, Charles F. Scott, Sydney Wilmot, H. R. Buck, A. B. McDaniel, and R. L. Sackett.

Robert Fletcher,† M. Am. Soc. C. E.—In general, the principles and methods set forth in the papers on Engineering Education, and the suggestions offered for securing a broader and more thorough training at the start, have been followed for about thirty years at the Thayer School of Civil Engineering, in Dartmouth College. Indeed, the beginning was made fifty years ago with a six-year course.

Professor Swain's view of the subject of engineering education is most admirable. The speaker has been fighting for nearly fifty years, as a teacher, against this idea of premature specialization; it is wrong. He has endeavored to make the young men under instruction see the essential unity of the fundamentals that dominate the entire curriculum that the would-be civil engineer should pursue. Mathematics, physics, mechanics, and their immediate applications in the science and practice of engineering, constitute a permanent and indispensable foundation, the breadth and depth of which cannot be diminished. The task is to give a man such a thorough grounding in those fundamental principles, and their applications, that he will not fail to see how they underlie and direct all that comes afterward. The increasing host of modern developments seldom presents anything new in principal. Even the almost phenomenal achievements of to-day in engineering practice mostly exemplify principles that were learned long ago, sometimes, it is true, expanded and extended as the result of research in our own day.

F. B. Sanborn,‡ M. Am. Soc. C. E.—The speaker will discuss only one phase in the new methods of engineering instruction, namely, that which concerns the fundamentals of engineering education. What are the fundamentals? Shall there be one list of fundamentals for an engineering school that is giving a practical education and another for an engineering school that is giving a theoretical education? A variety of lines of progress should be available, and a student should be able to select the school that is best adapted to his needs. One type of student might require for his best advancement that the practical be presented first, whereas another might require

* This discussion (of the Technical Papers on Engineering Education presented at the Annual Meeting, January 17, 1923, and published in *Proceedings* for March and April, 1923), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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exactly the opposite. The courses of many engineering schools embody two years of theory, followed by two years of applied science. The fundamentals in such a course commonly include algebra, geometry, trigonometry, calculus, physics, chemistry, English composition, English literature, history, French, German (or one other language); but if the curriculum is arranged for practical courses to come first, the fundamentals would include field surveying practice, practical laboratory tests in physics and chemistry, mathematical problems associated with their field and laboratory tests, likewise English composition, including expositions and letter-writing based on observations made in their studies. Also, the beginnings of courses in finance and economics might be given as fundamentals. The more advanced courses in mathematics, literature, history, and language would be given during the last two years of study.

The speaker contends that the theoretical-first method need not be applied to all students; in fact, his experience as professor in four different universities and colleges has proved to him that many students advance more satisfactorily by studying the practical demonstration first.

The importance of a new alignment of fundamentals was emphasized a few years ago by Professor Mann in his report to the Carnegie Foundation. He made an exhaustive study of engineering education, in school, among graduates, and among employees of graduates. As a result of this study he emphasized the human element, as a feature that should be developed in engineering education. He placed character building ahead of even computations and design. Character building was a new fundamental. It could be instituted in a college curriculum only by displacing some other line of study. What study should be displaced to make room for the study of character building? As Professor Mann's report became digested, the officials of many institutions saw that, if a real course of study in the human element were to find a place in the first two years of college study, it must displace fundamentals which must be moved to the later years and perhaps not be classed as fundamentals.

The establishment of courses in character building has progressed considerably since that report was presented. There is still opportunity for wider introduction of these fundamental subjects. The speaker has found it possible to introduce character-building courses in definite forms, not by giving such studies as incidentals and small parts of other courses, but as major parts of definite courses.

Various studies may be included properly under fundamentals. Each institution should be urged to select and to specify the courses of study that it proposes to furnish. Many college catalogues are too sweeping in their claims. It is not practicable for an ordinary college to give a course of study that is well adapted to the students who have a practical bent of mind and also to those who have theoretical inclinations. An institution cannot cover all the fundamentals and all the branches of the Engineering Profession in other than a cursory manner.

It would be better, say, for one institution to furnish instruction in the fundamentals of practical engineering; another to teach the fundamentals of

a theoretical education; and a different institution to qualify its graduates primarily for positions that combine business with engineering. Progress is a necessary step in present-day engineering education. These papers, the remarks of other discussors, and Professor Robinson's book "Mind in the Making", all point to the need of changes. It is a sign that indicates growth and activity. As new methods are considered, it is hoped that a new alignment of fundamentals will be emphasized; that there will be different types of engineering schools, some with practical courses of study first, followed by the theoretical; others with the theoretical courses first, followed by the practical; and that each school will state clearly and definitely what type of training it offers, and thus guide students in selecting a training which will be the most advantageous.

The speaker seconds the criticism by Mr. McCrosky. Such criticism refers specifically to the objection of requiring too much in each course of study.

J. K. Finch,* Assoc. M. Am. Soc. C. E.—The papers presented by Professor Scott† and Dean Raymond‡ emphasize strongly the fact that engineering instruction should consist largely of broad and thorough training in the fundamentals. In past years, there has been a tendency toward so-called vocational instruction which will produce graduates of high technical proficiency, will make well trained draftsmen and designers, but will not make engineers. Dr. Fletcher has stated that instruction at the Thayer School of Civil Engineering at Dartmouth College has always been based on the plan of thorough training in the fundamentals. The engineering schools of Columbia University were founded in 1864 on this same plan, and the course in each branch of engineering has always included thorough instruction in the sciences, mathematics, physics, chemistry, mineralogy, and geology, as well as in the fundamentals of the other branches of engineering.

In addition, Dean Raymond's paper discloses clearly the growing demand not only for this thorough and fundamental type of engineering training, but also for a broader kind of training which will fit the future engineer to meet the requirements of the rapidly broadening field of engineering effort. The engineer to-day is being called on to meet new problems in new fields where the engineering viewpoint and engineering methods are found to be necessary. Indeed, such opportunities are, in general, more attractive to young men of broad training and high ambition than more strictly technical positions. The paper shows that a careful analysis of conditions in the West argue for a five-year program, allowing time for some of the non-technical subjects such as English, history, and economics, as the best means of meeting this demand. About ten years ago, it was concluded that the proper solution of the problem of a liberal engineering training was to require engineering students at Columbia to take a college course before their engineering studies, as had already been successfully done in the courses of law and medicine. Those

* Associate Prof., Civ. Eng., and Director, Summer Session, Camp Columbia, Columbia Univ., New York City.

† *Proceedings*, Am. Soc. C. E., March, 1923, p. 492.

‡ *Loc. cit.*, p. 498.

in authority do not advocate that kind of a demoralizing country-club college course which Professor Swain denounces in his book, "The Young Man and Civil Engineering", but they do feel that the ultimate solution of the problem of a broad training for the engineer will be to base, as they have already done, his engineering training on a good sound college course. They do not think that this plan can or should be followed in every institution. Columbia, which is a great privately endowed institution, and which, therefore, is free to do what is felt to be best in advancing the standards of higher education, has met this demand in a manner that is thought to be the best and which will ultimately be so regarded. As an ideal it is certainly worth working for. As a practical plan it may be so far in advance of general engineering and public opinion that no institution can afford to make it obligatory.

What has been the result of this change, which lengthens the period of university study from four to six years? As far as attendance is concerned, the result has been just what was expected. The new plan went into effect at a most unpropitious time, but even with the conditions growing out of the World War, general business depression, etc., the attendance is about half the normal capacity and is increasing. It is believed that the ideal of a broadly trained engineering graduate is growing in the United States, the papers presented on this subject prove this, and that, in due time, Columbia University will receive the full support which its program merits. Certain of its requirements have doubtless been excessive and its procedure may have lacked in flexibility but, with modifications in details, those in charge are prepared to carry out this plan which they believe to represent a high ideal in engineering education.

The other result, which was also expected, is that the students entering the engineering school are more mature in mind, and more thoroughly fixed in purpose than those in the four-year course. Although the engineering course is in no sense a graduate engineering course, it is possible to pass over the elementary points in instruction much more quickly with these men than with the usual engineering student, and to devote more time to discussing matters about which there is room for a difference of opinion, for careful analysis of assumptions and practice, in short to emphasize the development of an "engineering mind", rather than merely dwelling on facts and standard methods.

There are two other points on which the speaker would like to comment: First, Professor Scott has spoken of the necessity of training men for certain positions. The experience at Columbia is that this cannot be done in a university. Not only must university instruction be broad, fundamental, and thorough, which will crowd out much special training, but that graduates often take up branches of engineering other than those in which they majored in college. Mining engineers become civil engineers, for example, and quite recently a Columbia civil engineering graduate was appointed to the presidency of a large company, his two predecessors being Columbia men, one of whom, however, was a mining engineer and the other an electrical engineer.

The other point which would seem to be of vital interest to the Society, is that if an engineering graduate goes into the broader field of engineering

effort, as many are doing, in which engineering furnishes a background rather than the main subject of his efforts, in most cases he no longer considers himself an engineer. There have been cases, for example, of Columbia graduates having gone into bond investigation work for banking houses, who have either never joined the Society or have given up their membership, perhaps to join a banking association. If such positions offer great opportunities for engineers and if, as seems to be true, it will be the duty of future engineers to meet many problems in other than strictly technical fields, it seems obvious that a determined effort should be made to keep in the Society these men who are carrying engineering methods into new fields which, if not strictly engineering from the technical standpoint, are nevertheless engineering from the standpoint of method, thought, and action. A large part of future engineering will be in this direction, and it is the duty of the Society to claim this field and to keep closely in touch with it. Many technical contributions will be found in *Proceedings,* but little has been done either to interest and to hold engineers who are in these modern fields of engineering work or to advance the application of engineering methods in other than strictly technical lines. This is an opportunity for the Society to increase its interests and activities and to hold in its membership an important group with which it is in a fair way to lose contact.

George F. Swain,* Past-President, Am. Soc. C. E.—There seems to be an idea—and it is well founded—that something is wrong with education. It seems to the speaker that one trouble is with respect to the fundamental question as to what education is. He has referred to this in the book mentioned by Professor Finch. Recently, President Hadley,† of Yale University, although he, perhaps, had not seen the speaker's book, has taken that idea as a text, and has written an interesting article, entitled, "What Is Education?"

Education, in reality, means a drawing out, an effort to develop the native, inherent, potential possibilities of the student. It has been largely interpreted, of recent years especially, to mean "to put in". The student thinks he is going to school to open his mental mouth and be crammed with information. That idea is wrong, and will never lead to good results. The object should be simply to train the mind of the student by proper subjects of study and proper methods of instruction. He should be taught the fundamentals of science, mathematics, mechanics, and certain branches of engineering, also, some history, English, economics, and languages, and should become possessed of a mind trained so that he may be able to turn it to the consideration of any given subject, and bring to bear his knowledge of any or all fundamental subjects that he has acquired. The tendency has been in the other direction. The field of science has increased enormously within the last few years; new subjects have been developed—the automobile, the tank, the aeroplane, radio—but they all depend on the same fundamental principles most of which were known years ago. Instruction in those specialized branches should not be given in the undergraduate course. Only the fundamentals that have been

* Cons. Engr.; Prof. of Civ. Eng., Harvard Univ., Cambridge, Mass.

† *Harper's Magazine,* December, 1922.

named should be taught, and the student should be allowed to come back for graduate courses, as has been stated, if he wants to pursue specialized branches further.

The tendency to give information rather than training has gone back into the preparatory schools, and the curricula of those schools are being crowded with subjects which do not belong in them. When the speaker was a boy, it was said that the child was taught the "three R's". He does not know what they are taught now, but he does know that they do not get the "three R's", and that even college graduates do not have them. They have been treated to a superficial taste of many different subjects, generally without learning any one of them thoroughly.

If educators could get back to the fundamental idea that they want to draw out innate power, that they want to train, instead of to put information into the student's head—although, of course, incidentally, they inculcate a great deal of information—that the main idea should be the training in fundamental principles, they would observe great improvement. The speaker knows that college graduates do not get such training to-day.

How far afield we have gone in engineering education is illustrated by the fact that Professor Turneaure's suggestion, which Professor Raymond has quoted,* and which is excellent and sound, is put forward as something new. Professor Fletcher has stated that it is what they have been doing at the Thayer School of Civil Engineering, at Dartmouth College, for thirty years, and these very principles have been applied at Harvard University and the Massachusetts Institute of Technology for more than thirty years. There is nothing new in them; they are fundamentals; they are important; and they should be acted on more than they are.

C. M. Spofford,† M. Am. Soc. C. E.—The speaker is heartily in accord with those who have expressed the opinion that the education of the engineer should be devoted primarily to rigid training in fundamentals, rather than to the effort to give the student a mass of information concerning practical details which are continually changing and knowledge of which can be acquired with much greater advantage in practice than in the schools. These fundamentals should include English, economics, the sciences underlying engineering, and the specific theoretical subjects on the application of which his particular profession depends, these latter subjects being accompanied by practical examples sufficient to illustrate their application, and given primarily for that purpose, rather than for information.

Inasmuch as the character, judgment, and vision of the engineer, as well as of other professional men, are of supreme importance, the desirability of developing these qualities should be borne in mind in all his training; and his various courses should have among their most important purposes the development of thoroughness, self-reliance, honesty, and imagination. It is the speaker's belief that the usual subjects in the engineering curriculum are well adapted to the development of such qualities, and that the exact subjects

* *Proceedings*, Am. Soc. C. E., March, 1923, p. 503.

† Hayward Prof. of Civ. Eng., Mass. Inst. Tech., Cambridge, Mass.

selected are of less importance than the logical arrangement of their subject-matter, the method of teaching, and the personality of the teacher.

DUGALD C. JACKSON,* M. AM. SOC. C. E.—The papers by Professors Scott and Raymond, and the discussion by Professor Fletcher might be taken to indicate that the engineering schools had been preaching certain doctrines in regard to education for thirty years, but have received so little acceptance that it is necessary to repeat them. This seems to be unfortunate; but the papers by Messrs. Alexander† and Harrington‡ indicate a different situation in regard to the acceptance of the doctrines referred to, and Professor Fletcher is to be congratulated that he was one of the early teachers in engineering to emphasize the ideas which the engineering schools now fully support, and which are receiving the endorsement of the Engineering Profession at large. It takes a long time to bring a general recognition of the appropriateness of educational doctrines. The problems are complex and difficult. There are no means of securing a specific or a formula which will determine the education that should be provided for a particular line of endeavor. There are certain threads, however, which experience has proved must be followed.

There has been a vast change in the attitude of the Engineering Profession toward the engineering schools during the thirty-two years in which the speaker has been a professor in an engineering school. When, as a rather young man actively engaged in engineering affairs, the speaker went to an engineering school as a professor, one of his intimate friends, also actively engaged in engineering affairs but considerably older, referred to the employment as an engineering teacher as "retirement to the cloister". That represented fairly the view in the Engineering Profession at large at that time. The graduate of the engineering school was looked on with suspicion, in the Engineering Profession, as a man probably of erratic tendencies, theoretical but not practical, and there was still a discussion of the relation of theory to practice. Since then, all this has been changed. It is now recognized that the engineering schools are not cloistered institutions for a few, who perhaps may have some curious ideals, but are the substantial seat of training for the present and future members of a great profession.

It is often said as a pleasantry, that the only profession in which fathers recommend their sons to follow their footsteps is that of engineering. Whether or not this statement is correct, it is a fact that the sons of a great many engineers enter their fathers' profession, and we must see to it that they have the proper education to enter that profession on a level with its present elevation and accomplishments. Also, whether our successors are our sons or the sons of others, we must see to it that the education of our successors is adequate. We must also distinguish carefully between professional education, which must be conferred on our successors and is to be provided in the great engineering schools, and vocational education, which is needed for a large group of employees in industry who do not reach the professional elevation. This is a matter of great moment to be considered and reflected on by the National Engineering Societies.

* Prof. of Elec. Eng., Mass. Inst. Tech., Boston, Mass.

† See p. 698.

‡ *Proceedings*, Am. Soc. C. E., March, 1923, p. 506.

During the past thirty years the medical societies have taken a greater interest in the education of the physician than theretofore; their interest was followed by rather active work in education for the law by the Bar Associations of the country; but it has been only during the last twenty years that the National Engineering Societies have taken an active interest in professional education for engineers, as exemplified in the performance of the engineering schools.

The first joint meeting of a National Engineering Society (the American Institute of Electrical Engineers, in that instance) with the Society for the Promotion of Engineering Education was held in 1903, and at that time there may have been great doubt as to the interest of the members in such a meeting. However, it proved so satisfactory that a similar joint meeting was held in 1912, with great satisfaction. In 1921 there was a joint session of the American Society of Mechanical Engineers with the Society for the Promotion of Engineering Education.

These joint sessions are very serviceable. They are encouraging to the teachers, and they should be illuminating alike to the teachers and the engineers in practice outside the colleges. It is to be hoped that many more meetings of this kind can be held, as the teachers receive very definite inspiration and encouragement, the engineering schools are guided to a clearer vision, and the Engineering Profession at large gains through the solidarity secured by the interchange of ideas between the teacher and the practitioner.

At one of the early joint sessions the speaker ventured to suggest that the engineer's education should be founded on the concept that an engineer should be defined as a man competent to conceive, execute, and administer great works, but this was then considered by others as rather an idealistic view. However, there seems to be an agreement that that is what we are now substantially looking for as the premises on which to build professional engineering education. The question arises: how can the situation be improved, and what can the engineering societies do in further aid? Mr. Harrington has made some definite and important suggestions, and some concrete and desirable proposals in respect to engineering students. He has suggested that there is more in the way of co-operation that can be accomplished. The foundation is now laid so well that it might be practicable for the boards of government of the several National Engineering Societies to consider this question and establish a joint committee for the interchange of ideas between the National Societies on this important problem. Such a committee would be serviceable in co-operating with the new committee of the Society for the Promotion of Engineering Education referred to by Professor Scott, and also in co-operating with the important committee on the education of professional engineers which the National Industrial Conference Board has convened.

The National Engineering Societies might also add their influence to broaden the activities of the Society for the Promotion of Engineering Education. Such support of the effort to co-ordinate more adequately engineering education on sound premises, which Professor Scott recommends so heartily, will go far toward establishing the study of the situation which he recommends, and which is recommended by the new committee of the Society for the

Promotion of Engineering Education. It is necessary to demonstrate that the Engineering Profession believes that such a study is desirable for the benefit of the Profession and the industries of the country, and of the social structure of the Nation, but with that demonstration the necessary impetus will be furnished, and the speaker is satisfied that this important study will be secured. It is to be hoped that this joint session may bring the matter to the attention of professional civil engineers so adequately that the Society may join in supporting the effort which may prove of such great moment in advancing the status of the Profession.

THEODORE T. MCCROSKY,* ESQ.—In view of the fact that education is being discussed, it is perhaps fitting that a student should participate to some extent.

In engineering education there is a definite tendency to overload the curriculum, and this re-acts to deaden the initiative of the student. There are so many fundamental courses which must be included, and so many cultural and practical courses which should be included, that the student who does justice to all has little time or energy left to devote to individual work. A student's time is monopolized to such an extent that he cannot pursue the particular subject in which he is especially interested. His initiative, which prompts him to do unrequired work in some branch of his profession, is stifled by the mass of stereotyped work required by the faculty. In order to satisfy his initiative along special lines, in order to live up to his ambitions, he must neglect his assigned work. Consequently, one hears, over and over again, the question: "Do we have to know this?" If this question were asked only by the undergraduate of the type who looks on his university as a country club, it would not justify mentioning, but, on the contrary, it is asked repeatedly by men who really come to college for an education. This attitude is far from praiseworthy, yet, in a sense, it is necessary.

There is no doubt that the faculty knows best what should be taught. Each student is faced with two courses of action—either to neglect in part his curriculum in order to devote himself to original work, done on his own initiative, in answer to the call of his ambition, or, to do his curriculum work as it really should be done, and stifle his initiative and the promptings of his ambition. Which is the better engineer—the man with plenty of initiative, but weak in his theory; or the man who knows his theory, but has no initiative, nothing but the ability to do what he is told to do? Certainly, neither is the ideal engineer.

This is not a new dilemma, but is one which should never be lost sight of. Attention has been called to it, because it has not been raised thus far, and surely, no discussion of engineering education would be complete without some mention of it. An engineering student, therefore, appeals to the great educators of the country to keep it always before them, lest the initiative of their students be killed.

T. CHALKLEY HATTON,† M. AM. SOC. C. E.—A subject in which the speaker has been interested for many years, and has discussed before many universities,

* New Haven, Conn.

† Chf. Engr., Sewerage Comm., Milwaukee, Wis.

is the education of young men so that they can express themselves before committees and boards in such a way that they will be understood.

The speaker does not know the number of hours for the study of English required by the average engineering school or college, but he does know that at present he has about fifty-five young engineer graduates on his staff, each of whom sends in reports weekly—sometimes daily—of his operations. These men are frequently called on to prepare specifications for their particular work, and it is astounding how little they know of the English language and of the legal side of public work and of contracts. The speaker has also presided over many engineering meetings and societies of young men, and has called on those whom he knew understood the subject under discussion, and has been embarrassed in finding how poorly they could express themselves. It seems that, although fundamentals are wanted—they are necessary in all engineering education—something broader is needed. A man can learn the fundamentals of engineering, and still be graduated a very narrow-minded man. He must know how to express himself in good English. He must know something about politics, that is, he must be in close touch with what the people want, must know how to find out what they want, and how to express himself in discussing their wants with them.

If universities would give more attention to the teaching of English, the speaker thinks that engineers would make greater strides than they are making or ever have made.

J. C. Ralston,* M. Am. Soc. C. E.—It is with hesitation that the speaker ventures any comment on so complex a subject as engineering education. He has often thought that there should be some orderly and more human method of classifying students before they are enrolled either in the engineering colleges or the universities, or that there should be some form of probationary enrollment based on something more than mere mathematical equipment—something more effective in human classification than that of continuing the present rather loose and inconclusive custom of accepting the applicant who satisfies certain pedantic requirements. Why should an engineering college enroll a pupil whose temperament is anything but that of the engineering mind?

Could not the matriculant be shaken down through a stack of psychological screens, from the coarse mesh of ineptness to the fine 200-mesh of specific qualities, in the same metaphorical manner as the metallurgist classifies the mechanical properties of his field samples before submitting them to the fire for final measure of worth and value? The speaker has seen scores of young men who have been trained in college as far as the colleges and their splendid staffs have been able to train them, but the students did not have the "engineering mind." It would be impossible to produce engineers from such material. The Engineering Profession has adopted incomparable refinements in its laboratories and elsewhere in determining qualities and qualifications of the materials of construction.

* Const. Engr., Spokane, Wash.

Have the colleges and universities met the concept of this challenge in their selection of human materials? The forces and materials of Nature are the alpha, and the agencies of human nature are the omega of engineering. Do the demands of college enrollments, blinded by the false notion of mere numbers, smother an understanding of the necessity for the selection of proper human materials?

This subject has been visualized before by many earnest and accomplished educators. The speaker is aware that the land grant colleges may be unable to enforce desirable restrictions, other than loose educational ones. Possibly even the less hampered universities have endowment or legal restrictions which administrative officers construe with a meticulous nicety; yet the challenge to the engineering educators and to the whole profession as to the selection of the engineering mind still stands.

It is granted that there is no super-man who may say that John Doe may not ultimately make a good engineer, but the speaker is constrained to think that the engineering genius in education is not so devoid of intuition, so lean of devices, nor so hindered with traditions, that it cannot use some of the methods or devices chosen by business and industry in selecting employees on whom special training is commercially worth while. In the speaker's judgment, this constitutes a challenge to the profession.

Another outstanding challenge to the broader concept of American engineering education is the preparation of the young man for the wider activities and duties of the future engineer. Technical education is well understood and digested, but the vision of applied ideals in engineering is overlooked or too vaguely understood.

What class of professional men in the United States has shaped the nation's religious ideals and traditions? Manifestly, those of the divinity.

What professional group has shaped and guided the nation's political ideals and traditions? What group has inspired the best political development, has constructed the fabric of the nation's highest political and legal aspirations, and made of them a national tradition? It has been the legal group.

Again, what professional group has shaped or is shaping the nation's industrial ideals? None. Industry in America, as far as it has gone, has accomplished superlative scientific and mechanical developments. Our industrial ideals are compounded only of cold material accomplishment. This, however, is only one-half the nation's best aspirations and ideals. The other half, the spiritual or human half, has been left untouched, except in so far as it has been poisoned by the virus of class-consciousness, super-unionism, and grudging service, directed by scolding, whining, and fault-finding groups of doddering agitators. These are the groups and types that are misshaping our real fundamental industrial ideals. The breath of constructive and patriotic genius has been smothered because engineering vision has been microscopic; the view has always been a "close up". The broad perspective has been lost.

What professional group, therefore, is qualified by training, education, and vision to direct and inspire the nation's industrial ideals and shape them into a great national tradition as the other two major national traditions have been shaped? What group has been remiss and has failed in the discharge of a

plainly obvious duty, or has failed to prepare itself for its broader activities in its own special field? Conclusively, the engineers.

This, then, is the other challenge—the challenge to leadership in industry, to the Engineering Profession, to the teachers of engineering. Let the challenge be met so that it may no longer be said that the spiritual vision of the Engineering Profession is charged with the exuberance of promise and the sterility of performance.

MILO S. KETCHUM,* M. AM. SOC. C. E.—For some time there has been an attempt to select men for the Profession of Engineering, and many have thought that it is possible to determine early in a man's career whether or not he will later show proficiency in engineering work. Since returning to the University of Illinois, the speaker has been strongly impressed with the records of two men whom he knew in their student days. These men were students in mining engineering, and were about the two most unpromising candidates for the Profession of Engineering that could be imagined. Based on present standards for rating men, these two were lacking in practically every essential that would appear to qualify them for the Profession of Engineering. One of these men was graduated, and the other "quituated". The former, at a later date, received a Ph. D. degree from a prominent Eastern educational institution, and is now one of the leading mining engineers in America. The other has made an excellent record in mining engineering. Students of the caliber of these two men would not at present be permitted to remain in any self-respecting institution.

Early in his teaching career, the speaker thought it possible to determine whether or not a young man would make a success as an engineer; but, during twenty-five years of experience he has been unable to find any method of selecting young men for the Profession of Engineering. There are so many elements that go to make up a successful engineer that no method of rating has been discovered which will be of any material assistance in determining whether young men should enter that profession.

Referring to the matter of the curricula of engineering colleges, the speaker has felt that perhaps we are losing our heads in our desire to make a man out of a boy—to get accomplishments at 21 that we should only expect at 40. It is difficult for a young man in college to be both broad and deep. Some months ago in talking with an alumni committee that was trying to determine how a certain course in engineering should be given—that this should be put in and that should be put in—the speaker finally said to one of the men:

"If all the courses that your committee is suggesting were placed in the curriculum, the student graduating would be very much like the Platte River. The Platte River is a mile wide and two inches deep; it goes wandering all over the plains; it sticks itself into everybody's business; it drops things here and there; and it accomplishes nothing. It is just a pestiferous nuisance. If you were to confine the waters of the Platte River within a narrow channel, it would be a self-respecting and useful stream; its velocity would be increased, and it would carry away the sediment in place of depositing it along the way."

* Cons. Engr.; Dean, Coll. of Eng., and Director, Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

In a professional way, most of us are inclined to be like the Platte River. We cannot be both broad and deep, and the speaker has come to the conclusion that we should give the most attention to making the engineering student deep, paying considerable attention to his ambitions and aspirations, giving him enough breadth so that he will finally become a useful citizen and be able to accomplish something in his chosen profession; and in that way we shall be doing the most we can hope to do.

The speaker has recently been much impressed with the lack of training of many men who have been engaged in research work. The records of all societies are cluttered up with the discussions of many subjects and much experimental work, not only useless but vicious. Tests that have been well thought out have been carried on, in many cases, by men without the necessary underlying theoretical training to realize what was intended to be accomplished, the result being that their data have been worse than useless. It is important that research work should not only be outlined by competent men, but that the details of the investigation should be carried out by men thoroughly well-grounded in practical and theoretical applications.

In addition to having been an engineering teacher, the speaker has had considerable experience in practical engineering in contracting work, and in engineering administration. The men in the speaker's employ who have given the best results have been those who made the best records in college and had a thorough theoretical training. These men were able to take up the business and practical side of engineering much more quickly and efficiently than those who had had superficial training along these lines. The broadening and humanizing value of any subject is largely in the teacher. After all, it is the great teacher—the man behind the gun—that is going to count; and it is not so important whether he teaches one subject or another if he puts the punch into his work, if he teaches the young fellow to be awake, to "get into the game", and to keep on growing.

W. E. WICKENDEN,* ESQ.—The speaker has been much impressed by the fact that a unity of objectives is sought, although through a diversity of agencies. The possibility of the Society for the Promotion of Engineering Education, as a liaison body between the great groups of engineers in the National Societies, is an idea that is growing in thought and attention.

The speaker will say little more to-day than to second heartily the suggestion that possibly it is time for each of the great National Societies to appoint a representative to confer with the committee that is already established by the Society for the Promotion of Engineering Education and the National Industrial Conference Board. The speaker has a strongly growing feeling that we are face to face with the question as to whether the lines of cleavage now found in the great engineering fraternity may not have possibly outlived a great deal of their usefulness. Did they not have their roots in an historic situation? Did they not originate in the evolutionary conditions by which each branch of engineering practice has differentiated itself from the parent stock? Are not the activities of engineers, especially those who share in the great industrial

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enterprises of the country to-day, subdivided along functional lines rather than on lines of historical evolution of the great engineering societies? If we have this question before us, certainly a body which is interested in the subject of engineering education has a great purpose to serve as a liaison body by which the aims, the ideals, the objectives of the great National Engineering Societies may be brought out of their present state of diversity and into a state of unity, as we approach our common objective of increasing the thoroughness of fundamental processes of education by which men are made ready to perform their tasks in the world, and carry on the processes of their own education after they are graduated from college.

Charles F. Scott, Esq.*—With the general purpose of indicating the evolution which has taken place among engineers, along the lines indicated by Mr. Wickenden, the speaker will discuss student branches, and may indulge in some personal reminiscenses.

Twenty years ago, the speaker was President-elect of the American Institute of Electrical Engineers, and made it a point to ask many of his colleagues what the Institute should do, and what its function was in the electrical engineering field. One of the replies was this: "I don't know what the electrical industry is going to be in the future, but in the past it has been doubling every five years".

The speaker knew that the progress of the past had been rapid, but, not having thought ahead, was startled by the idea that in a short five years we were likely to have doubled what then seemed a very wide use of electric service. Now, if in five years the volume of electrical industry was to be doubled, and in ten years quadrupled, where were the men coming from to handle the great task? From the schools? Then, to be an instrument in the development of electrical engineering, the National Institute should direct its attention toward the schools, in order that the students might be able more quickly and more effectively to take up the burdens of engineering work after graduation. That thought was followed by correspondence with several engineering professors—this was while the speaker was in industry, before becoming a teacher—and, as the outcome of the replies and a recommendation to the board of directors, student branches were authorized, and, within the next few months, were formed in a number of schools. Others followed, until now there are sixty or seventy. Some two or three years later, the mechanical students in one of the colleges, desiring to follow the example of the electrical students, formed a branch and made application to the American Society of Mechanical Engineers for recognition. A few years later, the mining engineers formed branches, and some two or three years ago the civil engineers started branches.

The idea of a common co-ordination between engineering societies with relation to student activities is so far in advance of the ideas of twenty years ago, that it has come to the speaker with somewhat of a shock; but, in the excellent arguments that have been presented, there are many reasons in favor of engineering students joining together in common effort in student activities.

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Whether this had better be one group, with subdivisions, or several divisions co-ordinated or federated together into a general group in a school, or whether all the students in common, irrespective of the courses they may be taking, had better belong to one general engineering society, the speaker is not yet ready to say; but the coming together of the engineering societies in one building, making it a center of common engineering representation to the students of the country, would be a fine example of co-ordination and one of the best things that the engineering societies could do in influencing the students in the colleges.

SYDNEY WILMOT,* Assoc. M. Am. Soc. C. E.—The plan proposed by Mr. Harrington, and amplified by Professor Scott, seems to involve joint activity by the Engineering Societies in pursuit of student chapter activities. This is one of the most important phases of our professional contact with engineering schools, and deserves the heartiest sentiment in its support.

There need be no argument as to the mutual advantages of student membership to the student and to the societies. In certain cases, however, when confined to a particular society, even such a simple relationship is almost impossible. Take, for example, a case of personal observation and interest (Brown University) in which the course of study involves three years of unified work by all students, and a branching out or specialization in the fourth. Thus the student need not decide definitely on his own peculiar trend of study until his senior year. In fact, he would usually hesitate to align himself with any one particular undergraduate engineering chapter. Likewise, a similar inhibition would operate in small schools where the total of students in several classes might not provide the necessary minimum number of men interested.

At present there are almost a dozen senior students under the speaker's instruction who might properly in the future become members of the American Society of Civil Engineers. They would gladly join now in some kind of co-operative effort with the Society as student members, if their number warranted such a step.

Obviously, in this case, the present plan is not advantageous for the needs of the situation. Only some kind of joint organization, as proposed among the several societies, will overcome the difficulty. Under this arrangement, all men in the engineering courses, whatever their year or their aim in engineering work, would feel that they were eligible. With such support, the organization might reasonably anticipate success.

At least one other National Society is considering the advisability (if it has not already taken action) of permitting the affiliation of any engineering student or group of students. Joint action, it would seem, is a far better solution.

The sentiment in favor of such a plan seems to justify it, and it is to be hoped that the Society may see fit to advocate it, as it will be an effort of potential value to the Society and of considerable possibilities to the smaller schools.

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H. R. BUCK,* M. AM. SOC. C. E.—Mr. Harrington has reported a condition among some of the student branches of the Engineering Societies that is discouraging and may become more so; but he has also presented an interesting picture of an honorary engineering society which would draw together the separate student branches in each of the colleges and technical schools, and which would serve as a common feeder to the four Founder Societies. Is it not practical to ask, what engineers are going to do about it?

Will not individuals take advantage of the opportunity and write to the Board of Direction, asking that a committee be appointed to try to obtain joint action with the other Founder Societies in improving the work of and the conditions surrounding the several student branches?

A. B. MCDANIEL,† M. AM. SOC. C. E. (by letter).‡—There has been considerable discussion in recent years among engineers and educators as to the curricula for the training of engineers. All kinds of combinations of general and technical subjects have been proposed in courses of four to eight years in extent. Many leading educators have strongly urged the modification of the present engineering curricula so as to reduce the number of highly specialized subjects and to provide opportunity for a broader and more fundamental training. Some engineers have recommended that such subjects as business administration, finance, accounting, and public speaking be taught in engineering schools in order to furnish a training in the business phases of the engineer's work. Whatever the plan proposed, or remedy suggested, it is evident that there is a need for a careful and thorough study of existing conditions. Professor Scott has presented the project of the Society for the Promotion of Engineering Education. The purpose of the writer's discussion is to present briefly another project concerned with the study of engineering education.

Experience in army training has shown that the first requirement for effective work is the preparation of detailed statements of exactly what the men must be trained to do. By thus clearly stating the objective for training, it is possible to produce courses of study which are much more effective than those produced under the old system in which specifications are not prepared. Industry has recently recognized the need for such occupational specifications, and many organizations are making the necessary job analyses.

Objection has been made that it is impossible to state the things an engineer must be able to do, and those he must know. Army experience has shown otherwise, however, and has demonstrated the need and value of descriptive statements of the qualifications required for various engineering positions. To meet this need, the American Association of Engineers has appointed a committee of engineers and educators, of which the writer is Chairman, to make a study of the classification of engineering positions. The Committee is confining its attention at first to the highway field, and is securing data from about 3 000 highway engineers in the service of the U. S. Bureau of

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‡ Received by the Secretary, January 22, 1923.

Public Roads, and six State Highway Commissions throughout the United States. Specifications for the various positions involved will be prepared from the data received. These specifications will be of service to employment services, personnel departments, civil service commissions, and similar agencies in the selection, assignment, and promotion of engineers, and will furnish information to educational institutions for the preparation of engineering curricula.

R. L. SACKETT,* M. AM. SOC. C. E. (by letter).†—There are two factors which cannot be overlooked when the subject of engineering education is treated fairly.

First, the average engineering student of to-day lacks experience, and, on the average, probably possesses less than that held by those who were graduated ten or twenty years ago. Furthermore, much of the education of the engineer will necessarily follow his graduation, even if five- and six-year courses are adopted. The great school of experience is a finishing college, and nothing can ever supplant it. This is no excuse for ineffective educational methods, but is only a fact which cannot be ignored in judging the efficiency of engineering schools and colleges.

Second, the importance which a student attaches to, say English, by way of illustration, is dependent on his home training, his association with engineers, if any, and the English used by his engineering instructors in regular class work. The importance that other students in his environment attach to English is determined in a similar way. Instructors are employed who were graduated some years ago, but had their experience under engineers who did not exemplify good English. These instructors are sometimes deficient in their working knowledge of good expression, either oral or written. This is also true of instructors in other than engineering subjects. No discussion of engineering education is complete if it does not realize that the salaries paid to engineering instructors are inadequate, and do not encourage a prospective engineering teacher to take a four-year liberal arts course and then a technical training in addition. More mature men are needed as teachers of freshmen, not only in teaching English, but as instructors in all subjects. Although habits of language have already become more or less fixed, they are more flexible during the freshman year than later. Better salaries would make it possible to employ men of broader training, larger culture, wider experience, who not only would teach the student but would also exemplify good English and careful analysis, and unconsciously deepen the student's appreciation of other important factors in his early education.

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† Received by the Secretary, February 5, 1923.

ENGINEERING RESEARCH

Discussion*

BY MESSRS. CLEMENS HERSCHEL, WILLIAM G. ATWOOD, H. H. ROUSSEAU, W. C. CUSHING, CHARLES RUFUS HARTE, F. E. SCHMITT, and HENRY GOLDMARK.

CLEMENS HERSCHEL,† PAST-PRESIDENT, AM. SOC. C. E.—The speaker has considered himself, in some respects, a little out of tune with some parts of what is called research; and, therefore, the following ideas are stated diffidently, and for consideration. He has always been guided in his work, by aiming at engineering utility.

There is a distinction between research, merely for the sake of research, and engineering research, so much so, that the speaker fears there has been a misuse of the word, "research". There is a difference between making experiments, of a kind that may be likened to amusing oneself, and an engineer's work in that line, which aims at a definite object. This may be illustrated by the work of Pasteur, who was recently commemorated, on the occasion of the 100th anniversary of his birth. Pasteur was a man whose life work was far removed from engineering, and, although called a scientist, and essentially living the life of a scientist, with his whole career devoted to science, he never did a thing that had not a definite, useful object for its aim. At one time, it was the cure of rabies; at another time, it was the prevention and cure of anthrax, in order to save the lives of sheep; or, again, it was the removal of a pestilential parasite from the vineyards of France, as well as a great many other such kinds of work. For years, his aim invariably was toward a definite object, and one that was useful to mankind.

It has always seemed to the speaker that every investigator, every one working in the so-called line of research, should criticize himself or question himself, and in this way endeavor to follow aims of the kind referred to. The speaker does not propose to sit in judgment on the work of those who are acting as investigators; he wants them to do it themselves, to themselves, to feel that what they do is for the purpose of advancing the cause of mankind, not merely to find out something new and record it in a scientific paper or in a book. That kind of work was useful in its time; but it has been multiplied; it has increased to such an extent, and there is so much buried in those books and in the transactions of so-called learned societies that, if a man wants certain knowledge to-day, it is just as easy, or easier, to go out in Nature and, by experiment, seek, and thus get it, as it is to search for it in the libraries.

* This discussion (of the Technical Papers on Engineering Research presented at the Annual Meeting, January 19, 1923, and published in March, 1923, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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What the speaker has said tends to distinguish the work of engineers from what is mellifluously called research in "pure science" and is claimed as the only original source of advance in the arts and sciences. He is questioning the validity of this claim. It may surprise many to be told (it is not the commonly accepted or popular creed, indeed it goes against it) that useful inventions frequently, perhaps generally, precede their scientific explanation, or alleged foundation. For instance, the speaker has doubts whether Marconi reasoned from Hertzian waves to wireless telegraphy. It is much more likely that he experimented with electrical apparatus, succeeded in sending messages through the air, and, later, had all this explained and computed by a consideration of Hertzian waves. It is certain that all the beneficent inventions of Edison were made without a knowledge of "pure science".

The engineers of America have had a great opportunity for the advancement of engineering knowledge afforded them by the foresight and generosity of an eminent member of their profession, when he created Engineering Foundation. Let it not disappoint the cherished hopes of the engineers of the country, by a perversion of its funds, in doing work other than that of engineering. The scientists of the country have had their particular work amply endowed. One of them, in charge of a huge endowment of this kind, some years ago, laid down the rule and criterion that the funds of which he had charge should not be used for anything that had a useful end for its aim and object. Engineers should be prepared to accept his challenge and are already stating that Engineering Foundation should have for its rule and criterion the opposite view, and should support nothing that has not some object that is directly intended to be useful to mankind. Thus will be maintained the engineers' desire to be of service, rather than to follow only a more or less selfish quest of pleasurable emotions.

WILLIAM G. ATWOOD,* M. AM. SOC. C. E.—The various research projects have been outlined quite fully by Mr. Flinn,† but it may be possible to add a little information concerning the work of the Committee on Marine Piling Investigations. This Committee was organized as a result of the recommendations of the San Francisco Committee which was formed on account of the total destruction by marine borers of many harbor structures in Upper San Francisco Bay during 1919 and 1920. The San Francisco Committee found that the field of investigation could not adequately be covered by a local committee and, to be successful, the work must be done on a National scale. The problem was to devise means for protecting wooden structures from marine borers and to find the causes and remedy for the deterioration of concrete in and near sea water.

The most urgent phase of the problem seemed to be a study of marine borers, and this was, therefore, the first one investigated. It was found that there was little knowledge of the distribution of the various species of borers or of their capacity for destruction. In order to obtain accurate information

* Director, Committee on Marine Piling Investigations, National Research Council, New York City.

† *Proceedings*, Am. Soc. C. E., March, 1923, p. 518.

along these lines test boards were placed and are being maintained by the railroads, various Government bureaus, municipal bodies, and industries owning property on salt water. The first of the boards was placed in June, 1922, and the last one only recently, and they have already yielded so much information that within a short time it will be possible to discontinue a number of them. Many new species of borers have been discovered, some of them with great power of destruction, and some previously known species have been found to have a capacity for living under conditions which had been thought to prevent their existence.

Good creosoting has long been recognized as the best method of protecting timber, but, in some localities, even well creosoted timber seems to have comparatively short life. The U. S. Forest Products Laboratory and the Chemical Department of the University of California are both making chemical studies of creosote and methods of impregnation, hoping thereby to enable the quality of the oil to be improved and the impregnation to be made more thorough. The University of California is also experimenting with several metallic salts which are locked in the wood by a process developed in the laboratory.

The Assistant Secretary of the U. S. Department of Commerce, by recent designation of the President, has become a member of the Committee, and arrangements have been made for a study, under the direction of the Chemical Warfare Service, of an entirely new series of poisons which may be injected into the timber. The Chemical Warfare Service will also endeavor to develop a method of poisoning the water somewhat in the same general manner that a gas barrage is placed on land. If this process can be developed, it will be possible to protect existing structures built of untreated timber when they are threatened by attack, and it may also make it economical to build structures of unprotected timber in places where borers are known to exist. Some of the methods for poisoning the water suggested will be practicable and inexpensive, if their efficiency is demonstrated by the proposed tests. The cost of this study, about $20 000 per year, will be paid by allocations from the funds of the various Federal Bureaus owning and maintaining structures in sea water.

The deterioration of concrete is as great a source of loss to property owners as the marine-borer damage to wood—if not greater—and because the use of concrete is increasing, the prevention of the deterioration is becoming of greater and greater importance. The records collected by the Committee indicate that there are few concrete structures in salt water, which do not show deterioration after a life of ten to fifteen years. Many of them show serious deterioration in even less time. The causes for this action are not known, but it appears that methods and materials that will give good results in fresh water will not do so in salt water.

It seems probable that the use of unsuitable aggregates, too much or too little gauging water, and improper mixing and handling, have been partly responsible for many failures, but careful study shows that chemical changes in the concrete are probably the principal causes of deterioration. These chemical changes may be greatly accelerated by the use of unsuitable construction methods or aggregates. Certain investigations of the Geophysical Laboratory

of the Carnegie Institution and of the U. S. Bureau of Standards seem to indicate the possibility of improving the cement itself. The Committee is planning a series of experiments based on the information gained from existing structures and previous laboratory studies, with the hope of finding the real causes of failure. After the causes are found, the remedies should easily be developed.

Both phases of the investigation are being based on the best scientific advice obtainable, as it is the feeling of the Committee that the solution of both must be based on pure science. For that reason, the membership of the Committee includes chemists and biologists as well as engineers, with the idea that only by the co-operation of pure science and applied science, represented by the engineers, a satisfactory solution can be found.

The Committee has had the enthusiastic support of the American Railway Engineering Association and the financial backing of the railroads. The Government support is coming largely from the Navy and Army Departments and from the Department of Commerce. The work thus far accomplished, has cost about $25 000 in cash and probably twice that amount in contributed services. During 1923, the cash expenditure will probably be $50 000 to $60 000, including the Government appropriations, and the contributed services will amount to at least one-third more than the cash expenditures.

The first preliminary report of the Committee covering only a part of the marine-borer phase of the study will appear in a forthcoming *Bulletin* of the American Railway Engineering Association. Further reports will be published as rapidly as the information developed will permit, so that engineers who are designing and building structures in salt water, may have the latest information available.

H. H. ROUSSEAU,* M. AM. SOC. C. E.—The speaker is in entire sympathy with Mr. Atwood and Mr. Herschel on this subject. From engineering research come progress and growth, and any means of furthering the ends of engineering science by promoting engineering research should hold a high place in the mind and attention of every engineer. The day has passed when civilization should be obliged to trust to the supplying of its daily increasing needs through the research work of individuals, working separately, each with more or less limited knowledge, resources, and facilities. This is the age of specialization and organization; it is very fitting, and accords with present-day thought and practice, that united and co-operative effort should be directed toward increasing the production of new ideas and processes in the practice of engineering, and accelerating the rate of development in all lines of engineering science—all for the betterment and advancement of the human race.

It is often claimed that military science and the implements of warfare develop during war at a rate one hundred times faster than during peace. Is not this a striking example of what can be accomplished by intensive, co-operative, research work under the urge of necessity? And who can say that mankind itself is not confronted to-day with a contest with Nature of urgency akin to an actual state of warfare, the outcome of which, ultimately, may

* Rear-Admiral (C. E. C.), U. S. N., Washington, D. C.

possibly determine man's very existence? Every day sees a diminution of our natural resources, many of which can never be replenished, and most of the activities of our every-day life are accompanied by glaring wastes that might be avoided. On every side we are aware of operations being performed at efficiencies that should be many times greater. The necessity for conservation and economy, by the human race as a whole, and for the adaptation and use of new methods and materials, is growing constantly greater. Should not the problem of providing for the manifold requirements of an ever-increasing population with diminishing resources be a spur as effective as a state of actual warfare to a realization of the necessity of a permanent campaign for carrying on intensive, organized, engineering research work in all its branches? And may not the results of such a campaign, directed by intelligent effort, and availing itself of 100% of the united ability, energy, knowledge, and resources of the engineers of the country, be similarly a hundred-fold greater than any progress that could be made by engineers working merely as individuals? No one would claim in war that soldiers fighting as individuals, even though much superior in numbers and equipment, could hope for anything like the measure of success that comes only through organization and discipline. Nature is jealous of her secrets, and organization and a well-directed plan of campaign are necessary to attain victory over her.

The Navy Department has been very glad to assist with all its resources the work of Engineering Foundation in investigating the attacks of the *teredo* on piling and timber, and determining the best methods of securing immunity therefrom; also the deterioration of concrete by the action of sea water. Both these subjects are of much interest, and any solution of these problems will be of great value to the Government.

W. C. Cushing,* M. Am. Soc. C. E.—Railroad engineers are much interested in almost any line of engineering research, and, as far as the speaker's connection with the Committee on Research is concerned, he does not know of any proposed research work that is not needed by the Engineering Profession. Such researches are not being conducted for the purpose of furnishing interesting study, but for the ultimate practical use which can be made of the work. The speaker, therefore, does not altogether understand the distinction between the two lines, that has been mentioned, and he certainly hopes that the opportunities for this kind of work will increase.

Charles Rufus Harte,† M. Am. Soc. C. E.—Engineers, particularly those who are in active work, are too apt to forget that what are called the laws of Nature are merely statements of what has been observed under certain circumstances, and though such laws may state exactly what occurs, they exist because of the occurrence; the occurrence does not follow because of the existence of the so-called laws, and in many instances, owing to the fact that the observations have not covered all the factors, the so-called "laws" are themselves incomplete or inaccurate.

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† Constr. Engr., The Connecticut Co., New Haven, Conn.

As any line of investigation, whether it is investigation in the sense used in the laboratory or is what is no less investigation, develops the continued production of something, there are found at all stages occurrences of various kinds which often seem to have no particular relation, or certainly no important relation, to the main work, but, as the development is carried on, it almost invariably results that new troubles or peculiarities are encountered, the explanation of which, in part or entirely, is found in this apparently unimportant and previously neglected occurrence.

Nothing illustrates this more clearly than the development of radio. The curiosity of Thomas Edison compelled him to ask why there should have occurred such an apparently inconsequential thing as the blackening of the tube of an incandescent lamp, not uniformly but locally; and it was the investigation of this, which, to his mind, accustomed to see in everything an indication of an underlying cause, resulted in the development of the thermionic bulb and all that depends on it.

Engineers should rid themselves of any mistaken idea that research is merely the closet pastime of the "theoretical college professor", using this expression in the mistaken sense that is attached so often to the word "theoretical". Research is "finding out", and it is of the utmost importance to the working engineer that every assistance be given to research work because, on the one hand, by the study of difficulties it serves to find the answer, and, on the other, by the study of peculiarities which are not at once referable to known facts it serves to broaden the field of knowledge, both in the working and the purely intellectual fields.

F. E. Schmitt,* M. Am. Soc. C. E.—Professor Talbot† and Mr. Flinn seem to have covered the subject very fully. One little difficulty, however, might well be examined and borne in mind. The essence of research, as the speaker sees it, is the discovery of something that we did not know before. On the other hand, engineering work is essentially a matter of working with tools that we have and that we consider satisfactory, and the common attitude of the practicing engineer toward his work and his problems is that the means at hand are ample—he is bound to take that view, or he cannot have confidence in the results of his work. The beginning of research is doubt as to whether our available means are ample. It is difficult for engineers—speaking primarily of civil engineers—to put themselves into that doubting frame of mind, and the speaker thinks that we must charge to this phase of the matter—in part at least—the difficulty of collecting the problems which face every engineer in his work, and of bringing them to a focus so that research can begin.

One associated point may be worthy of mention: The kind of research that Mr. Blackwell‡ has spoken of particularly—mass research—is, in a certain measure, applicable to civil engineering problems; but it has always seemed to the speaker that the majority of these problems relate to matters that individually are easily investigated. All over the country we have many

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* Associate Editor, *Engineering News-Record*, New York City.

† *Proceedings*, Am. Soc. C. E., March, 1923, p. 512.

‡ *Loc. cit.*, p. 548.

laboratories and many keen and enthusiastic men, who, if they can be put into contact with worth-while problems, will set to work to solve them. Research, however, is discovery. One cannot hire it, but must depend on spontaneous activity; and so the problem of stimulating a spirit of research in this Society, and thereby galvanizing its membership into a desire to advance the scientific knowledge of the Profession, depends on enlisting the individual effort of members all over the country, on stimulating a wish to join in the work of discovering new things and to put their individual abilities and powers into the general service. For that, again, it is necessary, first, to determine the individual questions calling for research, and then to bring them to the born investigators, those who have the capacity and the facilities. In this direction little has yet been done; and not much can be done unless the members generally appreciate the importance of this work and give it their co-operation.

HENRY GOLDMARK,* M. AM. SOC. C. E.—Relative to the admirable paper† by Mr. Burgess, on the work that has been done and is being done by the U. S. Bureau of Standards, the speaker recently had occasion to witness some of that work, particularly the tests in welding, and also the large number of tests on the duralumin columns used in the airship, *Z R-1*, now under construction by the U. S. Navy Department. The Bureau of Standards is not sufficiently supported; it is cramped by a lack of clerical assistance, and its work is limited by a lack of funds. Just what the Society as a whole, or its members individually, can do to induce Congress to make larger appropriations, the speaker does not know, but the members should use every opportunity afforded them to educate public opinion to the necessity of research activities, particularly those which are so admirably conducted by the U. S. Bureau of Standards.

* Cons. Engr., New York City.

† *Proceedings*, Am. Soc. C. E., March, 1923, p. 524.

CITY PLANNING

Discussion*

By Messrs. William T. Lyle, E. M. Walker, Charles W. Leavitt, Frederic A. Delano, Charles N. Lowrie, M. W. Weir, E. A. Fisher, Rudolph Hering, Harold M. Lewis, George P. Hemstreet, George A. Soper, Nelson P. Lewis, and Harold A. Caparn.

William T. Lyle,† Assoc. M. Am. Soc. C. E. (by letter).‡—That city planning is one of the greatest of modern movements is demonstrated by the fact that 50% of American cities of 100 000 people and more have city plans; that city planning has become increasingly interesting to the engineer is evidenced by the general trend of developments in the last three decades.

The fore-runner of modern city planning in America was the planning of the grounds, buildings, and waterways of the Chicago Exposition of 1893, which was a revelation to all those in attendance. The work at Chicago was largely responsible for the idea of "the city beautiful"; the dominant note was the esthetic, and to apply this idea was, for many years, the hope of civic zealots. Although the World's Fair provided a great impetus, it is also true that city planning would have been developed without it, for other reasons and by force of necessity. The phenomenal growth of cities is well understood by members of the Profession. At the beginning of the Civil War only 11% of the population of the United States lived in cities, whereas at present nearly 40% live in cities. This growth would have been impossible without improved facilities, railroads, rapid transit, the automobile, and the telephone; without them, modern manufacturing would have been impossible. Growth and complexity of development demand a city plan; the many evils attendant on an unregulated growth demonstrate its need. The great civic awakening of the last few years has disclosed many needs and opportunities. Business demands a city plan. Adequate transportation facilities must be provided. Conflicting railroad interests must be harmonized for the business welfare of the community. Grade crossings must be eliminated. Convenient and commodious wharfage facilities must be provided, with channels and harbors of sufficient depth for shipping. Building construction must be made to conform with modern hygienic requirements concerning light, air, and congestion. Throttled thoroughfares must be cleared and widened, boulevards provided for pleasure driving, service roads for trucking, and arteries of travel located

* This discussion (of the Technical Papers on City Planning presented at the Annual Meeting, January 19, 1923, and published in *Proceedings* for March and April, 1923), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, January 10, 1923.

along lines of least resistance. Parks and playgrounds must be established to satisfy the recreational and artistic requirements of the community.

The year 1909 is a memorable one in the history of the development of city planning in the United States, because in that year the First National Conference on City Planning was held. It is also memorable in England by the passage of the Town Planning Act. Since 1909, city planning has advanced rapidly and has established itself as a well recognized and highly important art; and this advancement has occurred notwithstanding the retarding influence of the World War.

Many attempts have been made in recent years to define this advancement. To make a composite of all such attempts would be impossible, so different and conflicting have been the conceptions of the framers. This advancement has been due not only to the fact that the problem has been studied from different standpoints—from that of the landscape architect, the economist, the sociologist, the city administrator, and the municipal engineer—but also because great progress has recently been made in the art itself through the making of concrete plans. In view of the increasing importance of the engineering aspect of city planning, the writer would define it as the art of systematically providing for the physical growth of a city and its environs. More particularly, it is concerned with the engineering problems of streets, parks, public buildings, transportation by water and rail, and the regulation of the use, height, and size of buildings.

City planning will continue to recognize esthetic requirements, but the problem has become so great that other elements, such as the utilitarian, the hygienic, and the economic, are now considered of greater importance. The emphasis has shifted, and the idea has grown by accretion. Modern city planning can best be characterized as an art comprehensive. It is concerned with interlocking interests which constitute a problem of the greatest intricacy, requiring a governing mind of highly developed visualizing power and coordinating ability. There are many matters in city planning concerning which the engineer is unable to speak with authority, such as legislation, administration, and esthetics, but in the main the writer believes that in each of its three phases—design, construction, and management—the problem is one of engineering.

The practical divisions of the subject of city planning have been presented by recognized authorities. Without attempting to discuss any one of these divisions at length, the writer will mention a few observations on each of them from his professional practice.

The park and parkway problem is assuming more and more an engineering aspect. The park problem of to-day is different from that of a few decades ago. It now constitutes an integral part of the comprehensive city plan. As the other factors in the problem, streets, railroads, and waterways, have a direct bearing on the park problem, so it, in turn, has a direct bearing on them. The great American municipalities were wisely guided when acquiring lands for park purposes. These lands were acquired while they could be had, before other operations and improvements and the rising cost of real estate rendered

their purchase impossible. Fortunately, much of this land has been left undeveloped. Other tracts have been improved, often at great expense, and converted into parks of formal or informal architecture, which are the pride of the municipalities which own them. The time has come, however, when many of these parks must be rebuilt in order that they may become increasingly beneficial to the public; in order that they may become useful in a maximum degree to all the population which they are intended to serve. It is obvious that hygienic, economic, and recreational desiderata will displace the esthetic to a considerable extent.

Regional planning is another evidence of the expanding scope of civil engineering. No city is sufficient unto itself; it is dependent for its existence and for its growth on the surrounding country and on other adjacent cities with which it transacts or should transact business. A regional planning which brings contact between producer and consumer and thus stimulates both production and consumption is a new undertaking, but one which will attain immense popularity. In regional planning the engineer will evidently occupy a prominent position, concerned as it is bound to be with considerations of water, railway, and highway transportation. He will also be called on to solve problems of water supply and sewage disposal as joint projects, to the great saving of each co-operating municipality.

Zoning is closely related to the major street and boulevard plan as functionally considered, and to the plan for parks and playgrounds. It permits the separation of different kinds of street traffic, and thus powerfully operates in the solution of the traffic problem. The chief cause of street congestion is the promiscuous intermingling of fast and slow vehicles. This is largely reduced through the operation of the zoning system. As the streets and arteries of travel are the principal concern, in city planning, zoning is very important. One of the chief objects of zoning is to stabilize real estate values, not only in the residential section, but also in the commercial and industrial sections. Regulative zoning is of such a nature as to offer the necessary facilities to business and manufacturing, such as transportation, pavements, sewers, and power. A sound zoning should be constructive rather than destructive; instead of merely operating to drive out objectionable plants and occupations, it functions rather in preventing their establishment in improper locations; and thus, indirectly, offers the facilities already mentioned. It is evident that a thorough understanding of the zoning problem necessarily requires an understanding of the nature of industry and transportation, and thus comes directly within the province of the engineer.

E. M. Walker,* Assoc. M. Am. Soc. C. E.—During the discussion of these papers the question of cost was brought up. City planning, as usually done, does not cost much, but if it costs anything, it should be justified. It is fair to criticize some plans that have been made because they served no purpose, and resulted in little tangible progress. Their cost, however small, may seem to have been wasted, but is it not true, in commercial and other private activities, that many plans and schemes are devised only to be dis-

* Grade Separation Engr., M. C. R. R., Detroit, Mich.

carded? It is sometimes worth a great deal to know what not to do. The final developments may include elements of many of the discarded plans. Engineers plan so that when they execute they may proceed in an orderly and economic, rather than in a haphazard, expensive, and "anarchistic", manner. No one can doubt that the population of the United States will continue to increase, and will require many things that do not now exist. Regional planning and zoning are to provide that these things shall be in proper relation to each other. The cost of such planning is small in comparison with the great losses caused by haphazard development. The correction of the mistakes of the past may appear to be costly, but even these can often be shown to be worth far more than their cost. The beneficiaries, whoever they may be, should carry the burden. The usual assessment method, or the excess condemnation method, will do so, if honestly and efficiently carried out. Whether or not the things shown on the plans are required should be referred to the beneficiaries for their decision.

The people should say whether they want such things as thoroughfares, parks, playgrounds, and public buildings specifically, and, in general, a community in which living will be worth what it costs, and not cost more than it is worth. Proper city planning cannot be accused of extravagance; but, to obtain proper planning, requires an enlightened public opinion and "eternal vigilance."

CHARLES W. LEAVITT,* M. AM. SOC. C. E.—Mr. Lewis† has featured the objects of the Committee on the Plan of New York and Its Environs, of which Committee Mr. Charles D. Norton is Chairman. It seems most fitting that Mr. Lewis should be connected with this Committee because, for many years, he has taken such a prominent part in the city's accomplishment along these lines, and is known to have her best interests generously at heart. It is highly gratifying to know that he is going on with this work, using his intimate knowledge of the conditions which obtain here; he will be able to guide the efforts of this Committee to success in their greatly needed undertaking.

Mr. Lewis states that the Sage Foundation, which is helping to finance Mr. Norton's Committee, was created largely, if not wholly, for the improvement of social relationships and better living conditions among the people in New York City. It would seem as if this were indeed the whole object of the presentation of these papers and discussions. The greatest difficulty in the work of engineers who have devoted years to solving city planning problems is that, in helping with the enormous undertakings of routine, they often lose sight of the extremely important fundamental principles which are the cause of the work. They find themselves so involved in the tremendous detail of the betterment of environment that they temporarily ignore the primary matter, which is the simple fact that actual living in that environment must be bettered, or their effort is futile.

Regional planning, so-termed, may be a new topic to some engineers who have not been following the more recent developments. Many cities in England have found it wise to endeavor to scatter population, rather than concentrate it

* Civ. and Landscape Engr., New York City.

† *Proceedings*, Am. Soc. C. E., March, 1923, p. 554.

at certain points. The results of concentrated population in New York and in some other cities in the United States are now seen, and it is realized how much better it would be for the lives of the people if they were not subjected to such conditions as exist, for instance, at the corner of Fifth Avenue and Forty-second Street, New York City.

In planning these districts, or regions, the idea is to make them, as far as possible, independent and self-contained, so that they may function in themselves and thus avoid much interurban traffic. This seems to be logical, and has the further advantage of giving the employee the opportunity of living somewhere near his work, possibly walking to it rather than traveling in a crowded carrier. Regional planning very happily might include the distribution of park areas between the various regions, so that children may get to the parks without traversing extensive built-up areas.

The questions of water supply, sewage disposal, and other municipal services may be carried on much as they are now, by laying mains of the proper sizes between the regions; or it may be advantageous to provide separate plants for each region. This is a matter to be determined for each individual case as it comes up for consideration.

One of the greatest arguments in favor of regional planning is that it provides greater opportunity for character development than is found in congested centers; there is developed more individual interest in the neighborhood, more pride in appearance, and a feeling that the individual voice carries weight and can obtain results; the individual citizen is not last in "the maelstrom", as he is in the center of large cities.

In the United States, property values have been rising and falling in a way to make investments in realty speculative. Realty values on Broadway, below Fourteenth Street, New York City, have gone up, then down, and to-day are advancing. The same holds true of the Twenty-third Street District. At present, great wealth is being poured into the Forty-second Street District. If the congestion which now threatens grows materially worse, these values may fall. It is similar to any other speculation, in that, when a certain point is passed, there is a danger zone, and the investor must look out. Such concentration of values can be ameliorated, to some extent, by regional planning, which should appeal to the investor, even if matters of health, right living, and esthetic development do not arrest his attention.

Mr. Fisher has spoken of the work of the New York State Association of Real Estate Boards. Mr. Childs, who took active part in the work of the Committee on City and Suburban Planning of that Association, helped greatly in the work done by that Committee last year, when an ordinance was framed making it necessary for either residents or landowners of a community to file with their local government any and all plans for the development of property, for the purpose of providing that arterial ways, parkways, and other public passages should not be stopped by private, arbitrary development, planned without consideration of the good of the whole community, whether such community be considered as village, city, county, State, or Nation. An ordinance on such a basis means much for the future. It is acknowledged that

developments must be controlled by legal authority, or they will form obstructions. Through every section there should be an unobstructed thoroughfare for travel, particularly in these days when competition makes it necessary for the individual to transport himself and his stock in trade from one place to another in the shortest time possible.

The ordinance, which was in the form of a resolution when passed by the New York State Association of Real Estate Boards, is now being framed as a bill and, in due course, will be presented by the Association to proper legislative bodies for action.

As horses have disappeared, to a great extent, with the exception of their continued use on bridle paths, etc., some changes must be made in the design of highways, parks, etc., to accommodate the motor-driven vehicles which take their place. There should be no grade crossings where it is possible to avoid them. The traveled way should be treated more or less as a railway, as the speed has been advanced to 10, 20, and 30 miles an hour. The occupants of cars can observe only a small part of the landscape, and the scale of park treatment, lawns, planting, fountains, statues, and other features must be on a larger and coarser scale, or the whole picture will appear as a blur, rather than as a reality; the areas taken for development must be large, so that objects to be observed can be placed in proper perspective.

It must be remembered that the automobile has made it necessary for modern man to live his life at least four times faster than was customary before motor cars appeared. He dashes from place to place, crowds four days' activity into one, and then wonders why he feels tired and confused. The parkway cannot escape; it must be put into scale and viewed as a moving picture film.

It is not advocated herein that landscape design should suffer by the production of plans in coarser detail, but the scale is becoming colossal, as may be observed almost anywhere throughout the country, because of the necessity of being in harmony with the wide and ever changing vistas viewed from quickly moving cars. For instance, some gate piers were designed on which, at close range, the sculptured details were charming and complete. To the pedestrian, passing by, the charm still remained, but, to the occupant of the automobile, going at the rate of even 15 miles an hour, it was blurred, and high relief had to be used in order to bring out the design satisfactorily.

A story is told of a traveler in England who started his automobile trip at the rate of 25 or 30 miles an hour. After traveling the first day, he attempted to record what he had seen, but soon discovered that he had seen a great deal and observed very little. Subsequent experiment proved that satisfactory observation could be made at a speed not exceeding 15 miles an hour. If the enjoyment of design is to be for automobile passengers traveling at their present rate of speed, design will have to be less intricate and more clearly defined than is now usual, or no enjoyment in it will be possible.

Where adults and children wish to walk and play, automobile drives are not now desirable. Such drives should be fenced off and protected, like railways, if they must exist. They should be made distinct from the parks, which are places of rest and relaxation, with playgrounds for little children.

It may be extreme to advocate fencing off automobile ways, but the number of children who run from pedestrian paths into such ways, and are killed there, is appalling. Drastic action should be taken to prevent this useless loss of life.

Arterial ways must be provided from cities into the country, and making proper connections between cities. As the automobile truck has developed into one of the chief methods of transportation of freight (the comparison is: waterways, 250 000 000 tons; railroads, 2 600 000 000 tons; trucks, 1 400 000 000 tons),* and has caused congestion and danger on the highways, it may be well to provide automobile truckways separate and distinct from the ways for other travel, exclude trucks from parkways, and, as far as possible, have the automobile truckways skirt rather than pass through—but with branch connections into—the towns and cities. Parkways, on the other hand, may very properly pass through the center of the town or city, and be made part of the interurban traffic route. It is remarkable to note the growth of freight transportation by truck, and realize that to-day it amounts to almost five times that by waterway, and more than half of that by rail.

Realty values along motorways have not advanced; such ways are not pleasant places of residence, although they are desirable near-by, for transportation purposes. It is doubtful if, at this time, there is available sufficient data to warrant the formation of an opinion as to just what is the reaction of automobile ways on adjoining realty values. On the Concourse, through the Borough of the Bronx, New York City, there has been a great deal of building. Whether or not residence there will be attractive permanently is a question, and, on the answer to this and similar questions, depends the future of automobile parkways in and through congested districts.

Although the speaker does not intend to discuss zoning, he wishes to call attention to one fundamental remark made by Mr. Knowles: "All zoning controls population". It certainly does. Whether that control is for right or for wrong remains to be seen. A zoning ordinance should be so broad as to permit of flexibility.

In the early years of the United States, the settlers had perfect zoning; they had plenty of room around the village green and they developed fine grounds which enabled them to have plenty of space where they could raise the right kind of children to populate the land. Whether or not it is right to permit the sort of building that is going on to-day around Forty-second Street, New York City, and whether or not it is right to restrict residence districts to a limited number of families per acre, are very grave questions, which only time will settle. It is gratifying, however, to hear as fine an authority as Mr. Knowles state that the number of people per acre should be kept down, and that no ordinances should be passed which cannot be regulated later, because people may find such ordinances very irritating, whether for or against them.

Mr. Knowles' paper† on the relationship between the engineer and the city plan is most interesting and to the point, but it does not bring out quite

* From address by Julius H. Barnes, Esq., President of the Chamber of Commerce of the United States, *Proceedings*, Am. Soc. C. E., for March, 1923, p. 132.

† See p. 713.

strongly enough the human element which has been developed very recently in the engineer. Although the engineer is necessary for city planning, city planning has been positively wonderful for the engineer; nothing ever has brought out the human side of the engineer as has present-day city planning. The city engineer and the practising engineer have awakened to the responsibility of their standing in the world; they find that they must appear before political bodies, meet where other people do, and must have at heart the best interests of human relationship, in order to make plans which will provide properly for right living. It has been a perfectly splendid incentive to the engineer, and has spurred him on to give to the public, in the best possible way, the benefit of his trained mind.

It is suggested that a city engineer should have very close acquaintance, always, with the city engineers in the towns nearest him; he should cultivate their acquaintance, know them, be fond of them, and endeavor to bring about proper connections between his city and theirs, thus making possible the closest suburban communication throughout the country.

Very recently the American Institute of Consulting Engineers succeeded in placing an engineer on the Board of Directors of the Chamber of Commerce of the United States of America. The advice given by this engineer during his membership on that Board has been appreciated greatly, and it is hoped that, on his impending retirement, another member of the Engineering Profession may take his place, carry on there the work which an engineer should do, and advise on important matters where his technical knowledge is needed.

FREDERIC A. DELANO,* ESQ.—The notable fact brought out by the discussion of city planning is the wide and general interest which obtains. Time was when this subject was confined to a specialized branch of the Engineering Profession, that of the military engineer. Now, however, it interests not only the engineer but every man concerned in civic development, not solely the architect and the landscape gardener, but the sociologist; and now it appears that all public-spirited citizens are actively aroused, and regard the subject as one of major importance concerning everybody.

The writer's interest was first directed to the problem in Chicago, and has been a subject of continuing interest ever since. The Committee of the Russell Sage Foundation, referred to by Mr. Lewis, is interested in the subject in New York, but realizes the great difficulties to be overcome and does not believe that any one man or even a small group could be found to prepare a plan for New York and its environs which would meet general acceptance. In an area of such diversity of conditions and interest, therefore, it would seem wiser to bring together the consensus of views and co-ordinate them if possible into a comprehensive whole.

In Chicago, of course, the problem was less difficult, and Mr. Burnham, by reason of his achievement with the World's Fair of 1893, had the confidence of the community to such a marked extent that he was readily accorded the lead, but even he saw the necessity of arousing the interest of a considerable

* Washington, D. C.

group of business men. He began by holding meetings twice a week in his "sky parlor" office overlooking the lake, and these meetings grew from a small group of perhaps half a dozen to four times that number. Finally, when the Plan of Chicago was made public and submitted to the city authorities, after two years of zealous work, it represented the awakened interest of at least one hundred of the leading business men of the city.

The next step was to secure general co-operation and adoption, and this was done by the selection and appointment by the Mayor of the city of a committee of some three hundred men representing every ward and section of the city. This committee, with such changes and additions as time has necessitated, is still, after 14 years, the guiding force behind the plan; but even this strong committee could not have accomplished much unaided, and the most effective assistance was secured by interesting the school children. This was accomplished by the preparation of a textbook on planning, which illustrated the work being done in other cities and that contemplated in Chicago. This booklet, put into the high school curriculum as part of a course on civics, awakened wide interest, and this was stimulated by prizes and commencement oratory.

The advantage of the plan was that it not only educated the coming voters, but through them did much to arouse interest among their parents and friends. This experience serves to illustrate the importance of a wide dissemination of knowledge in order to secure the co-operation of a large community.

CHARLES N. LOWRIE,* ESQ.—Changes in park design made necessary by automobile travel are extensive and interesting. They relate principally to roadways, grading, and plantations. Park playgrounds, which are neighborhood conveniences, and small parks or squares, which are usually in the central sections of a city, do not affect a discussion of the problem; but, as soon as larger areas are considered, such as Central Park, Prospect Park, Franklin Park in Boston, or others of that type generally throughout the country, the problem of automobile traffic immediately arises.

The basis of the change brought about by motoring is related primarily to the question of speed. Comparatively narrow roadways and park driveways were used with horse-drawn vehicles. Grades could be fairly steep, if necessary, and there could be reasonably sharp turns and curves. Narrow parkways, steep grades, and sharp curves, are equally difficult for motorist and pedestrian. In future parks of moderate size, it would seem desirable to minimize motor driveways as much as possible, and in some to transfer automobile traffic entirely to parkways or boulevards.

The questions of grades and especially curves, and the danger to pedestrians are obvious, and call for general reduction. If grades are steep, the noise is prejudicial to the quiet and reasonable peace of a park. If heavier cuts and fills are necessary in order to obtain better grades and curves, a disarrangement of natural conditions is produced, hence care should be taken to select as secluded a line as other conditions will permit. Sharp intersections should be minimized.

* Landscape Archt., New York City.

As to new plantations, there is again the question of speed as a governing factor. With horse-drawn vehicles one had sufficient time to observe landscape effects with more leisure and in more detail. At present it is better to design the landscape scenery, as far as possible, on very broad lines, including the planting. The beauty of intimate detail is lost to the motoring public.

If fewer motor driveways are placed in the parks, more parking places should be provided at the entrances, or, if in the parks, at points where large groups are likely to assemble, such as a waterfront feature, near a music pavilion, etc.

The speaker believes that the use of automobiles will do much toward linking isolated park areas into park systems by utilizing boulevards and parkways, with the parks abutting as much as possible and in some cases being elongated into a combination of parkway and park. By that process, the people who live in the immediate vicinity as well as the motoring public will have enjoyment of such parkways. Parkways are usually in the form of a circuit system, which is the most accessible type.

M. W. WEIR,* M. AM. SOC. C. E.—Papers on City Planning and Regional Planning nearly always lay particular stress on the neglect of communities to plan ahead so that their physical growth may be controlled and orderly. It is the speaker's belief that education in this subject should be given to the masses in such a way as to teach them clearly the penalties to be expected if neglect of advance planning is continued persistently, and to show how vitally business interests and tax rates are affected thereby.

Although experience is an able teacher, her fees are high. Especially is this true when her repeated lessons are allowed to pass unnoticed by the community at large and are realized only by a few public-spirited citizens who are willing to give their time to striving for co-ordinated action by the citizens of the community.

New York City has passed through a number of experiences which proper advance planning would have made easier, at least, if not eliminated entirely. Although New York City has made plans for her future, she even now is confronted with new problems which, in a way, are an outgrowth of the very plans already made, because those plans did not go into the possibilities of the future with sufficient detail, particularly in regard to transit facilities and the ultimate limits thereof.

At the moment, additional lines of transportation into centers of business activity in New York City without doubt will relieve congestion of transit leading to those centers, but, at the same time, it cannot but increase the congested condition of the streets in those centers. It is obvious that the increasing of street areas to any appreciable extent is out of the question, because of prohibitive expense and the consequent loss of area for building use. Such increase, if carried far enough, naturally would cause a total failure of purpose. Although New York City furnishes a good example of conditions resulting from years of previously unrestricted growth, many other cities manifestly are approaching like problems of no small magnitude,

* Cons. Engr., New York City.

and, if prompt action is not taken, some will be fit, if lamentable, illustrations of the lesson at hand. It has been said that it is advisable and necessary to by-pass through street traffic away from business centers. This attitude, though correct under ideal conditions, is in some respects open to objection, from the viewpoint of practicality. When the history of retail business is examined, it will be found that business, in search for success, has found satisfactory location on arteries of heavy traffic, in fact, it is now the practice of a certain chain of retail stores to determine their locations on the basis of the traffic passing the places under discussion.

An example of some value in illustrating this point occurred not long ago at Port Chester, N. Y., where the Boston Post Road—a through road of heavy and diversified traffic—becomes the main thoroughfare of the business district of the town. Because of temporary necessities of construction, for a time the through traffic was diverted to another street one block north. Almost immediately merchants on the Boston Post Road complained of loss of business and, as time passed and the idea of making the diversion permanent was suggested, they protested bitterly, stating that as much as 40% of their business in some lines was transient and derived from the through traffic on the Road.

To illustrate further the relation between successful business enterprise and traffic: There are points where transportation facilities draw large numbers of people, such as the Summit Avenue Station of the Hudson Tubes, and several points in the Bronx at or near transit stations. A few years ago these points were residential only, but now, since transportation facilities have increased the street traffic, important business districts have grown up and become ambitious to gain greater importance. Communities, as well as individuals, have ambitions, they have the need and desire to expand, to gain in proportions and importance. This is but natural and healthy, obviously it is folly to attempt to atrophy such growth or to throttle such ambition.

City Planning and Regional Planning, to be efficacious and successful, must show proper regard for the principles of political economy, which, of necessity, enter into the problem, and for the natural desires and ambitions of the people comprising the community affected.

E. A. Fisher,* M. Am. Soc. C. E.—The City Planning Bureau of Rochester, N. Y., has now been in operation for about 4½ years. The city has no City Planning Commission, as many other cities have. Its city planning is done by a Bureau in the Department of Engineering. One great advantage of this is that this Department is utilized in making the necessary surveys and maps, and it also puts the Department in close touch with what the Bureau is doing.

This Bureau is composed of two parts, a Superintendent of City Planning, and an Advisory Board composed of four citizens and the Corporation Counsel. There are no commissions in Rochester; all operations are under the single-headed department plant; thus the Superintendent takes the place of a commissioner.

* Cons. Engr. and Supt. City Planning Bureau, Rochester, N. Y.

The city has been able to secure the services of some of the most prominent and public-spirited citizens, those having knowledge of the needs of the city, as members of this Board. A former Mayor was Chairman for 4 years. The action of the Superintendent is presented to this Board, and, when approved, is final. Members of the Art Commission are serving as members of the Advisory Board.

As the duties of the Superintendent, as far as they are related to the laying out of streets and matters of that kind, are similar to those of a commissioner, it is not necessary to describe them.

With reference to zoning, the code provides only for zoning for "use". The experience in Rochester in relation to zoning was altogether different from that of Detroit. Public hearings were started in order to educate the public, as had been done in other places, but this did not meet with success, as very few people would attend. The matter was presented to the different civic organizations, and the general consensus of opinion was that the zoning should be carried on, and should have been done years before. It was concluded, therefore, to adopt zoning regulations. It was not necessary to have an ordinance, as is required under the General City Planning Act. The Superintendent made up the regulations for "use" zoning, presented them to the Advisory Board, and on September 22, 1919, they were adopted, put in operation, and have done a great deal of good. Since the zoning was put into effect, quite a number of people have found out what it means.

The Superintendent has stated publicly, as his belief, that any system of zoning that cannot be changed without too much trouble is worse than none.

There has been no general change in the zoning plan, but numerous small changes. It is necessary also to make exceptions. It was found in New York City that, in order to prevent undue hardship, it was absolutely necessary to create some board that could vary from the strict regulations of the zoning ordinance, and the Board of Standards and Appeals was permitted or authorized to allow special variations.

A Board of Appeals was not needed, because it was considered that the Advisory Board had the authority and could act as such a Board.

The speaker has noticed in the newspapers a decision relating to an exception made by the New York Board of Standards and Appeals. That is of much interest, because the same thing is done in Rochester, and, if the city did not have authority to do it, it would result in a great deal of harm.

The city authorities also believe in the employment of experts who have had experience in other cities, and do not believe in turning over the plan to experts to make up a plan and carry it out, as that belongs to the city itself. The city has employed quite a number of experts, eminent in special lines of work.

As the Common Council considered the height and area of buildings as a part of the Building Code, an amendment to the charter was secured which provided that the Common Council might enact an ordinance governing such matters.

The plan was prepared by the Superintendent of City Planning, and a first ordinance has been adopted and is now before the Council. The Act contained a special provision that no ordinance could be passed without the approval of the City Planning Bureau, so that "height and area" are considered with the "use".

This covers in a general way what is being done in Rochester. The ordinance has been in operation about 4½ years, and there have been no cases in the Courts. There has been considerable individual criticism, but the general feeling in the city is in favor of carrying out this plan.

RUDOLPH HERING,* M. AM. SOC. C. E.—With reference to the papers under discussion, the speaker would like to add a few historical remarks.

In 1887, the speaker was Chief Engineer of the Drainage and Water Supply Commission, in Chicago, Ill. While in that city he joined the Chicago Literary Club, and was asked to read a paper before it. Not wishing to present a purely engineering subject, and having made some recent studies regarding the natural development of cities, he selected a subject which he thought might be of some interest to the citizens of a large and modern city, especially one of rapid growth.

The theory of evolution in general, and particularly with reference to social conditions, had always interested him, and among others the natural growth of centers of population seemed to follow laws, the recognition of which could be helpful in understanding and furthering development. He had collected, where available, the plans of old cities in Europe and Asia at different stages of their growth. A set of plans of Paris from the Middle Ages to modern times was particularly interesting, and was given later to the Library of the University of Pennsylvania, where it presumably now is.

Literature on this subject had also been collected, partly from England, where the so-called "Garden Cities" had been developed more than a generation before. The best book on this subject was found to be one compiled, perhaps 40 years ago, by the City Engineer of Cologne, Mr. Stübben, who was later transferred to Berlin, and became responsible for some of the extensions of that city.

All this material helped the speaker to trace what seemed to be laws which the natural needs of cities indicated, and which appeared, at first automatically and then through engineers, to be utilized with benefit and economy to the citizens.

Much of the matter referred to in these papers was recognized or indicated in Europe a generation ago. Mr. Stübben speaks of the regional development and of zoning with the differentiation of occupations. The speaker referred also to the natural zoning that had automatically developed in New York City, because it was helpful to business, and pointed out the centers for banking, for wholesaling, for dry goods, jewelry, hardware, for amusements, for high, medium, and low-class residences, etc., already well differentiated at that time. L. M. Haupt, M. Am. Soc. C. E., in the Seventies, pointed out, before the Engineers' Club of Philadelphia, the advantages of street layouts with ref-

* Cons. Engr., New York City.

erence to economy of travel, advocating diagonal avenues to connect the various centers of business and residence, with reference to the saving of time, labor, and wear.

Such were the chief data for the Chicago paper. It received the usual polite applause, but the speaker felt that the subject was not yet fully appreciated, even in a city like Chicago, at a time when such a study might have been of great economic value. Of those present, only one gentleman, Mr. Burnham, the mention of whose name by Mr. Delano called forth these remarks by the speaker, made a few comments. In effect, he said: "That is all very well, but is beyond the appreciation of the present city officials and property owners." As architect of the World's Fair buildings and grounds in Chicago in 1893, Mr. Burnham himself showed in his magnificent designs and suggestions that he was already alive to the new movement in Europe, and since that time it has been quite naturally accepted also in the United States, and pursued by a number of able American engineers, and, the speaker hopes, with great advantage to the future development of the cities of this country.

HAROLD M. LEWIS,* M. AM. SOC. C. E.—In his paper on "Regional Planning",† Mr. Nelson P. Lewis has referred to the desirability of establishing a system of zones, or rings, of open spaces, which might help to prevent the enormous increase of congestion in metropolitan areas. Such spaces would necessarily bear a vital relation to the park and boulevard system, and one can readily imagine the whole area still further divided into sectors by a system of radiating boulevards. Two possible illustrations of this theory may be mentioned:

New Jersey has recently received a wonderful opportunity for establishing what might be called a part of the framework of such a system. Agreements were recently completed whereby the Morris Canal, which was chartered in 1824 for 99 years, will be acquired by the State of New Jersey which will thereby come into possession of a strip of territory about 90 miles long and averaging about 79 ft. in width. It illustrates the possibility that such a strip would have in breaking up the congested areas through which it passes, and the congestion that will undoubtedly develop along those parts of it which are now on the edges of the metropolis. At the same time, it portrays the difficulties of obtaining regional co-operation. Lately, many suggestions have been made for different uses of this land by the municipalities along its route for their own local projects, but it would be a shame if it could not be developed in some manner so as to give the advantages of its use to all the public in that part of New Jersey, as well as in the other parts of the metropolitan area of New York.

The second illustration has developed from a study of the facilities for recreation which has become increasingly important in the last few years as the stress and strain of modern business has increased. Table 2, compiled by the Physical Survey of the Plan of New York, is submitted in order to show the relation, or perhaps discrepancy, between those areas in the suburbs of

* Executive Engr., Russell Sage Foundation, New York City.

† *Proceedings*, Am. Soc. C. E., March, 1923, p. 554.

New York City now available for public recreation and those used for private recreation, as shown by the areas of golf and country clubs within the district. The figures have been computed for the public parks and the golf and country clubs in Nassau, Suffolk, and Westchester Counties in New York State, and also that part of Fairfield County, Connecticut, which might be included in New York and its environs.

TABLE 2.

	Areas, in Acres.		Percentage of Total Area.		Assessed valuation of golf and country clubs.	Ratio of club areas to park areas.
	Public parks.	Golf and country clubs.	Public parks.	Golf and country clubs.		
Nassau County, N. Y.........	70	3 400	0.04	1.9	$2 342 000	50
Suffolk County, N. Y.........	76	3 000	0.01	0.5	1 518 000	40
Westchester County, N. Y....	2 288	5 230	0.41	1.8	10 598 000	2.3
Part of Fairfield County, Conn.	714	1 360	0.27	0.5	804 000	1.9
Totals........................	3 148	12 990	0.21	0.86	$15 262 000	4.1

Deducting from the figures for Westchester County, the Bronx River Parkway, which is under the control of a State-appointed Commission, and the recently acquired Mohansic Park, Westchester would have only 288 acres of parks, and its ratio of club areas to park areas becomes 18 and the total ratio becomes 11. No parks of less than 1 acre in area are considered in Table 2.

The speaker believes these figures show, in a striking way, two things: the lack of park provision in these parts of the Metropolitan Area and the great demand for recreation facilities. These club areas are now important but temporary open spaces, and the suggestion is not a new one that they might readily be made permanent if some public authority, or commission, would acquire these parcels and then rent them back to the clubs for such a number of years as would correspond to the period that they would naturally stay there in any case. All these clubs are more or less temporary in character, and tend to move farther out from the city as congestion develops and the value of land increases. The speaker believes this possibility would be well worth study, not only in the New York metropolitan area, but wherever there is danger of over-congestion.

George P. Hemstreet,* Esq.—Regional planning is a splendid idea. Proper zoning undoubtedly adds to the value of the community, but, having made the plans for improving the outlying Metropolitan District, who is going to pay the bill?

The natural answer is that the tax payer will make the investment, but that the increased value of his property due to the improvements will repay him amply for the money expended.

The rent payer looks at improvements from this angle: Improvements cost money. Result: increased taxes. Final result: increased rents.

* Hastings-on-Hudson, N. Y.

In cities of the first class, the rent payer has some protection from the unscrupulous landlord, due to the emergency laws governing rentals. These laws do not apply to villages adjoining the cities of the first class, and as a result rents have been raised to such an extent in the villages of the Metropolitan Area, that people of modest means are becoming desperate.

Under the school laws, rent payers as well as property owners can vote on a proposition to raise school funds through a bond issue, and, in numerous instances, appropriations for increased school facilities have been defeated by the vote of the rent payer, who has been willing that his children should go on part time or be herded together in an unsanitary manner rather than have the rents raised again.

Mr. Lewis has mentioned the fact that in the Metropolitan District there are nearly 400 different municipalities. Regional planning in the Metropolitan District covering these numerous municipalities would undoubtedly meet with violent opposition from the rent payer when the time came to vote the money to carry out such needed improvements. These are hard, cold, disagreeable facts, and they must be faced.

Before additional funds can be voted for parks, sewer systems, broad avenues, splendid public buildings, and all those public things that add so much to the value of the community, it will be necessary to satisfy the rent payer that all these needed improvements are not going to be loaded on him, together with a handsome profit for his landlord.

George A. Soper,* M. Am. Soc. C. E.—The speaker would like to have one of those who have spoken for the Sage Committee on The New York Plan explain just what is meant by the use of the word "plan"? Some who have heard of the New York Plan seem to think that a scheme of physical development is contemplated, such as might be represented on a map or series of maps. Some think that the word "plan" is used to mean a scheme of development together with a statement of policies and procedures which should be followed in order to carry the physical plan into effect.

The speaker is sure that all are much impressed by the thoroughness with which the fundamental data necessary to comprehensive planning in the Metropolitan territory are being collected, but some do not quite understand what tangible form the Committee's conclusions are to take. It is hoped that it is not premature for the Committee to explain this matter.

Is the plan to be announced as a definite and complete thing at some future time, or is it to be a continuing project, which will be revised, extended, and developed for an indefinite period in the future, announcement of certain features being made occasionally as seems suitable?

There are many in the Metropolitan District who would like to be enlightened on these two questions, if it is not inconsistent with the policy of the Committee to make its purpose clear with respect thereto.

Nelson P. Lewis,* M. Am. Soc. C. E.—Mr. Soper has asked whether what has been called the "Plan of New York and Its Environs" is to be a concrete

* Cons. Engr., New York City.

and complete thing at any definite future time, or whether it is to be constantly revised and continued for an indefinite time. This is a difficult question to answer, as the speaker can make no statement concerning the policy of the Committee, and his active connection, as already explained, has been confined almost entirely to what has been described as the Physical Survey. To the present time, this has consisted chiefly of an inventory, or an attempt to visualize the problem. To this end, many maps and charts have been prepared, which can be seen at 130 East 22d St., New York City, and will be gladly explained to any who are sufficiently interested to call there. Although plans for a town, or a region, are sometimes referred to as complete or finished, there is no such thing. Conditions are changing constantly, and such plans can never be complete or final. They will be to a large extent suggestive only, and this is necessarily the case where hundreds of political units, in three States, are involved. There are many special lines of investigation which might be taken up and translated into definite plans by the officials of a particular city or town, but which, in the study now under way, can only be suggested as worthy of careful consideration by the local authorities.

In the speaker's paper, the need of regional planning and the attempts already made in this direction were pointed out. Such planning must be a continuous performance. No group or combination of municipal, county, or State governments, however harmoniously they might work together, could produce a complete and final regional plan. An unofficial group of citizens may be able to indicate to the local officials how the plans for adjoining political units may be made to harmonize with each other and fit into a comprehensive regional plan. If this is done tactfully, and if the suggestions are found to possess real merit, the work of the existing committee will have been justified.

Harold A. Caparn,* Esq.—The general problem of city planning resolves itself into one of transportation of man and his commerce. Even zoning is a part of this problem, because one of its purposes is to regulate transportation.

In New York City, the question of city planning at present seems to be one of building additional subways, so that it may be possible for more people to travel farther and farther from their homes to their business and back every day. This kind of program, however, cannot go on indefinitely. It merely results in making the city larger, more unwieldly, and more difficult in which to live and do business. Yet, like other large cities, New York takes a pride in her size, as Mr. Lewis had pointed out, which seems to the speaker somewhat like a man 5½ ft. high taking pride in weighing 600 pounds. The solution should lie, not in multiplying transportation, but in making it unnecessary, in studying how subways can be dispensed with, so that most men might be able to live within walking distance of their work. The speaker does not know how this could be done, but thinks that every city planner should work with this idea in mind.

Another subject that should be discussed is the condition of the city parks. Parks are a very important part of the city plan, but would fail in many of

* Landscape Archt., New York City.

their best functions if they were not maintained properly. Many of the parks in New York City are in unsatisfactory condition, and people are wondering why. The fault is not with the Commissioners, or the Mayor who appoints them, but in the method of administration, as this differs from that of any other park system with which the speaker is acquainted. Five Commissioners (so-called) are appointed by the Mayor every four years. They are mostly inexperienced in park affairs, and by the time they have acquired some education at the public expense, they are dropped; then, usually, five new and inexperienced men are appointed by the incoming administration, and the same process is repeated.

If one city department more than another needs continuity of policy in its management, it is that of the Park Department. Many years are required to make and develop a park, and, in New York City it would probably take many years under the best conditions of management to ascertain how to adapt it to the best needs of the varied and changing population and conditions. The parks should be administered by men trained in park affairs and secure in office as long as they make good. This principle should prevail throughout the whole organization. "Hiring and firing" should depend on merit only, not on political pull. In fact, the parks should be removed from politics and operated like any other large business concern. Yet a continuity of policy is made practically impossible by the vicious provisions of the city charter which relate to the Park Department.

The New York Chapter of the American Society of Landscape Architects has drawn up a plan for the reform of the Park Department which its members would be very glad to explain to this or any other body of citizens interested in city affairs.

SOME NOTES ON THE LOCATION AND CONSTRUCTION OF LOCKS AND MOVABLE DAMS ON THE OHIO RIVER, WITH PARTICULAR REFERENCE TO OHIO RIVER DAM NO. 18

Discussion[*]

By William M. Hall, M. Am. Soc. C. E.[†]

William M. Hall,[‡] M. Am. Soc. C. E. (by letter).[§]—Mr. T. P. Roberts[||] seems to think that an erroneous inference may be drawn from the paper, to the effect that his distinguished father not only recommended movable dams, but recommended the particular types of movable dams finally adopted. Such interpretation was not intended. To be more exact, on page 57 of the report of 1870 by the late W. Milnor Roberts, Past-President, Am. Soc. C. E., reference is made to a "plan of movable hydraulic gates for chutes designed by T. R. Brunot", and there are other references to movable dams in that report. On page 132 a definite recommendation as to the character of the improvement proposed states that:

"from Pittsburgh down * * *, slack water navigation would appear to be the most desirable, but for the reasons which are given in my study of the plan of locks and dams, I think that none but low dams with chutes should ever be placed there".

The writer believes it to be evident that he fully intended that the chutes should be closed with some type of movable dam. As narrated by Mr. Roberts, for 30 years or more prior to that report and recommendation, much interest had been manifested in the type of project for the improvement to be adopted, and several pamphlets were printed and distributed describing the proposed improvements and their respective advantages. The definite recommendation by W. Milnor Roberts is the earliest known to the writer for the design of dams with a movable part for the purpose of reducing the height of the fixed part. At about that time, Addison M. Scott, M. Am. Soc. C. E., under direction of the late W. P. Craighill, Past-President, Am. Soc. C. E., was commencing the improvement of the Great Kanawha River. He visited Europe, inspected the movable dams of France and Belgium, recommended the use of the Chanoine wicket for the dams of the Great Kanawha improvement, which were adopted by Maj. Craighill and the War Department, and commenced their construction in the early Seventies. A few years later, Col. Merrill adopted the Chanoine wicket for the movable part of Dam No. 1, Ohio

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[*] Discussion of the paper by William M. Hall, M. Am. Soc. C. E., continued from October, 1922, *Proceedings*.

[†] Author's closure.

[‡] Parkersburg, W. Va.

[§] Received by the Secretary, February 1, 1923.

[||] *Proceedings*, Am. Soc. C. E., March, 1922.

River. It is believed that the first Chanoine wickets built and placed by the United States Government were on these works. With that beginning, as stated by Col. Harts,* Division Engineer, Central Division (including the entire river), much work and study have been required to plan the project, with all its mass of details as it is now formulated. The writer has read many of the engineering reports on the river, made during the past 70 years, and fully concurs with Col. Harts in regard to the character of the improvement, and that the best, of the many types of construction proposed, have been selected.

The remarks on page 8,† in reference to the possible slow permanent growth in traffic, refer especially to package traffic by the public carrier, the "packet-boat" or "packet-line". On the upper river, where the channel and other conditions are favorable, barge-line traffic by privately owned barges and boats is already progressing under favorable commercial conditions, except for the interchange of traffic from barge line to railroad, which is usually believed to be with a discrimination against the barge lines. Therefore, the barge traffic by privately owned barges and boats should expand as the improvement progresses down stream, and it appears to be doing so. The statement by Mr. Roberts of the tonnage capacity of some of the locks is a valuable record. However, as it is believed to include only coal in barges loaded to capacity, it would hardly be wise to use it as basic data for estimating the capacity of the Ohio River locks for passing a mixed tonnage with possibly less that 50% of coal. The important thought which the writer wishes to convey, to those not familiar with the Ohio River improvement, is that it is converting a river with a past navigation of intermittent character and frequent annual or monthly suspensions to one which is continuous, reliable, and of sufficient capacity to admit of any probable growth or expansion which may be desired in case of war, railroad strikes, or the suspension of railroad growth, or in case of other local or National calamity. With this improvement completed and properly maintained, it will be only a question of providing the requisite boats and barges and manning them and possibly of building a second lock in 25 to 50 years. It is believed that it is the part of wisdom for communities, States, and the Government to provide reserve resources in transportation, especially when it can be done at such a small first cost, and by a method which involves a minimum of cost for operation and maintenance. A highway of equal extent, permitting an equally quick expansion in tonnage by automobile, or any kind of railway, held in continual readiness for use, will cost in annual upkeep probably ten times that of the Ohio River locks, dams, and channel. Although the United States has a wonderful railway system, one of the lessons of the World War was that the transportation system was insufficient for such an emergency, hence the great importance of providing the reserve in transportation afforded by the Ohio River.

The description of the industrial conditions in the Ohio Valley, together with the economies of the project by Col. Harts, is a valuable addition to the reference thereto on pages 7 and 8.† The opinions relating to river traffic are

* *Proceedings*, Am. Soc. C. E., April, 1922, p. 1007.

† *Proceedings*, Am. Soc. C. E., January, 1922.

somewhat divergent, and conclusive reasons for a failure to have a great increase already in the package traffic are so difficult to determine definitely, that it would hardly be advisable for the writer to dwell on them here, but he thinks the main cause for the failure, however, is stated in the paragraphs, near the middle of page 8.* The interesting discussion of the subject by Maj. Elliott,† with a statement of the traffic to June 30, 1921, and the economics of the project, is elucidating, and may satisfy the suggestion by Col. Harts on page 1007.‡ As the annual traffic through some of the locks in the Pittsburgh District has exceeded 25 000 000 tons, it appears reasonable to expect that, by the time the project is completed to Dam No. 48, below Evansville and Henderson, the average annual commerce for the Ohio River will be not less than 12 000 000 tons, with an average movement of at least 200 miles, or 2 400 000 000 ton-miles. Notwithstanding Maj. Elliott's interesting discussion, it appears that, after the effects of the swollen after-war rates of interest and prices are past, the annual interest on first cost, plus the cost of operation, care, and maintenance may be expected to be about $6 000 000. These estimates make the annual fixed charges per ton-mile for the entire commerce about 2½ mills. An examination of "Poor's Manual" does not disclose corresponding figures of average cost per ton-mile for railway track maintenance plus the interest on first cost of the completed permanent way; however, the average revenue per ton-mile for all the railways of the United States is given as 7.4 mills for 1920, 6.46 mills for 1919, 5.99 mills for 1918, 4.56 mills for 1917, 4.06 mills for 1916 (the year of low prices before the war), as compared with 6.62 mills for the years, 1918 to 1920, of swollen prices due to the war. Although the writer has not obtained the corresponding statistics for 1921 and 1922, it is known that the railway revenues have continued to rise, and when these statistics are completed, the average revenue for these two years will probably be nearly 1 cent per ton-mile. As it is believed that the average interest and dividend rate on railroad bonds and stocks does not exceed 5% annually and is probably nearer 4¼ to 4½%, it becomes evident that the railway rate for interest on first cost, which is believed to be about 2 mills, added to the maintenance, must be 2½ mills and probably considerably more, that is, as much as for the tonnage of the average railway of the United States as for the Ohio River, or more. As many of the railways in the Ohio Valley, under present traffic are working to their capacity, and as the 2 400 000 000 ton-miles for the river can easily be multiplied 5 to 10 times, when such quantities of tonnage are offered, it appears that, as a transportation line for the entire valley, the improvement is unquestionably advisable, and much more desirable than additional railway trackage of an equal cost. It is believed that a good double-track railroad for the entire length of the river, ready for operation, would cost 50% more than the completed improved river.

To the public, another advantageous feature of the improvement is that referred to on page 8,* "railroad low rates at competitive points", and more strongly emphasized by Maj. Elliott at the bottom of page 1284.§ It is so well known that water competition lowers railroad rates at all competitive

* *Proceedings*, Am. Soc. C. E., January, 1922.
† *Proceedings*, Am. Soc. C. E., May, 1922, p. 1281.
‡ *Proceedings*, Am. Soc. C. E., April, 1922.
§ *Proceedings*, Am. Soc. C. E., May, 1922.

points that it does not seem to be necessary to present more statistics or discussion. It appears that these advantages of emergency insurance in case of national or industrial war, and low freight rates effected by water transportation are not given due weight by many writers and speakers on this subject. The writer is so confident that this river will ultimately comply with the financial test enunciated by Col. Harts, Maj. Adams,* and Mr. Knowles,† that, in this particular case, these other great reserve advantages need not necessarily be used as governing factors. However, it is not clear to the writer that the test, as enunciated, is the one to be taken as the final mandate by the Government in all cases, because it is well known that the principal benefits of the railways of this country are not to the stockholders and bond holders but to the people who use them, who own property near them, and to the national body politic. The Government is now spending many times as much for highways and national paved roads as for rivers and harbors. No such test as that referred to seems to have been suggested as a condition for that expenditure. Why, then, should such a test apply and be the final criterion for river improvement any more than for public highways for automobiles, horses, and pedestrians from which no revenue is received or expected except in the way of national prosperity and the tax thereon?

In the movement of package freight, the writer believes that a change in the present system of operation must come before the volume approaches that which should exist and be expected for a proper service to the public. It must be understood that the building up of extensive service of that character for the public requires financial responsibility, an organized and permanent force of trained agents equal to that of the railways, with no possibility of "cut throat" competition, a continuity and permanency of service equal to that of the best railroads, some uniformity in quality of service and schedules of boats, and an interchange of traffic with the railroads, with one bill of lading only from shipper to consignee, whether or not the delivery is made by railway or by boat. With these defects remedied and such conditions made equal to those of the best railroads, there is a large tonnage of package freight moving by railways along the banks of the river which can be delivered by boats more economically and in less time than by rail, thereby resulting in economy and convenience to the public. Maj. Adams' remarks on this subject indicate that he may have overlooked the writer's suggestion that any award of special privileges should be under proper Government competitive methods and control. The essential accomplishment desired is that the "packet line" from the head to the mouth of the river establish and maintain a permanent service, for the country people, towns, and cities, equal in accommodation, and reliability, and superior in speed of movement to the best railway service. Until such a service is realized a lack of genius as a people to manage efficiently affairs of that character will continue to be displayed.

The writer appreciates Col. Brown's discussion‡ of the conditions in the lower river as a valuable extension of the brief reference thereto on page 9,§ especially as to the reasons being considered and the interesting features intro-

* *Proceedings*, Am. Soc. C. E., April, 1922, p. 1009.

† *Proceedings*, Am. Soc. C. E., October, 1922, p. 1706.

‡ *Proceedings*, Am. Soc. C. E., April, 1922, p. 1001.

§ *Proceedings*, Am. Soc. C. E., January, 1922.

duced by the omission of two or more of the dams below Dam No. 49. As he states, the short season of low water, the great fluctuations in stages, the great quantities of shifting sand, and the failure to find rock at any of the sites on which to found masonry makes the estimate of first cost high; and the shifting sand will no doubt make the cost of operation greater than that for those dams above Cincinnati. With the new 9-ft. channels through obstructing bars planned so as to have the direction of flow of the current parallel to the axis of the channel at all stages, the probabilities of success appear to be favorable.

In reading Col. Brown's statement (page 1005*), of the time required to place rolling gates in service after the winter floods, it should be remembered that, although two weeks' time is required at two or more locks below Cincinnati, the cases are exceptional. The delay in first maneuvers of the gates after floods is usually due to the accumulation of deposits of great depth on the gate tracks and about the gates. In the recesses which are placed so as to have a current flow over them at a rate equal to the average channel flow, the accumulation of deposit during one winter is not serious. The deposit in the recesses of Lock 18, after the three winters, 1910 to 1913, averaged 3 ft. in depth at the portals and 5 ft. at the rear ends; the maximum depth not exceeding these averages more than 1 ft. This small depth of deposit is due in part to the location of the lock on the concave shore. Where other conditions make it desirable to select a location on a straight stretch, or on a slightly convex shore, in order to obtain the swiftest current possible, the recesses should be as far out in the channel as other conditions will permit. A location directly below a jutting point, on a bar, immediately above a bar, or in a natural eddy is likely to cause excessive deposit. Deposit is not easily remedied by covering the opening. To be effective, the covers must form practically a perfect seal, and as they must be broken as soon as the flood is past, that method is expensive.

As Col. Brown states, in this long series of movable dams there has already occurred a serious shortage of water immediately after raising the dams, resulting in stages of possibly as much as 3 ft. below the natural stage for periods of several days.

The writer has read Mr. Knowles' report to the Pittsburgh Flood Commission, referred to in his discussions. As apparently anticipated in his discussion and stated by Col. Brown, it appears that "a comprehensive scheme of control of operating the dams will have to be devised", or, at least, that appears to be desirable. In devising a complete system of control, it may be well to consider the possibility of using stored water and of obtaining some of the supply from the pools of tributaries with fixed dams now in existence, and also the further possibility of increasing such storage from dams proposed for future construction on tributaries.

The system of steel sheet-pile coffers in use for the lock and abutment at Dam No. 34, to which Col. Brown refers, was started under his direction, for the purpose of reducing the great cost of coffers for work of this class, which, on the middle section of river, is nearly $200 000 for a single lock and dam. It

* *Proceedings*, Am. Soc. C. E., April, 1922.

consists of a series of small coffers ranging in size up to 49 by 150 ft., built of a single wall of steel interlocking sheet-piling about 40 ft. long, driven in rectangular or curved forms, and braced by several tiers of 12 by 12-in. timber. This method has been tried to a more limited extent at three up-river dams, and on many smaller industrial works. This is its first adaptation to all the parts of an Ohio River lock with a deep foundation of piles. The principal advantages of the steel coffer appear to be:

(*a*) It is not so likely to be injured by floods, and permits starting coffer construction a month or more earlier than with either of the wood types.

(*b*) Less excavation is required.

(*c*) Less damage to the coffer by floods and by deposit, and less machinery in the coffer to be submerged.

(*d*) Less obstruction in the river, thereby not so much increase in the velocity of the current, or obstruction and interference with navigation.

(*e*) The coffer wall is closer to the permanent work, thereby enabling derrick boats to stand outside the coffers and place loads in the permanent work.

The principal disadvantages are believed to be in the slower progress in seasons of long low water, excess in cost during seasons when there is no damage to wood coffers by floods, and excess in difficulty of removal, especially where piles are driven deep. It is also evident that the cost of the steel coffer construction increases much faster by increase in the span between the steel walls than the distance. This cost becomes excessive when the timber braces are longer than 20 or 24 ft., and require splicing or are purchased in unusually long lengths. A comparison of the cost reports of the building and removal of Lock 34 coffer and of those of Lock 38 box coffer, which was constructed and in use during the same time, indicates that the cost of Dam No. 34 is the greater.

Maj. Adams' data of the hydraulics of the river at Dam No. 48 (Mile 804) and his description of some of the main features of the dam are a valuable addition to the paper. However, when the dam was designed, about 10 years ago, it was hardly realized what a great improvement the improved bear-trap would make in the ease of regulating the upper pools, and the great reduction it effects in the labor of operation. In recommending that one of the Dam No. 18 bear-traps be placed next to the abutment, it was expected that part of the difficulty of passing from pass to wicket weir would thereby be overcome. As indicated on pages 22 and 39,* it has long since been clear to the writer that it would have been wise to have placed both traps together and adjacent to the abutment, even at the further expense of increase in the cost of bank protection. It is now gratifying to learn, from the discussion by Mr. Grimm,† and by Mr. Thomas,‡ that they also have reached the same conclusion in regard to the relative position of pass, wicket weir, and bear-traps for other dams now being designed or constructed. As the 91-ft. bear-traps can

* *Proceedings*, Am. Soc. C. E., January, 1922.

† *Proceedings*, Am. Soc. C. E., May, 1922, p. 1273.

‡ *Loc. cit.*, p. 1279.

be relied on to maneuver in one or two minutes, up or down, without any other effort than that required in opening two valves and closing two others, and, as the up-keep is small, it appears to the writer to be inadvisable to build a dam without one trap; two are preferable; and, as stated on page 40* and by Mr. Grimm, page 1277†, the excessive cost of the bear-traps is one of the principal reasons for not adding a third trap instead of 100 or 200 lin. ft. of weir wickets, as in the dams of recent design. However, another reason, which also must be considered, is the fact that a head of water is required to remove deposit from the trap recesses and inlet and outlet culverts immediately after the wickets are raised. The wider the bear-trap opening the longer the time needed to obtain the required head. At that particular time, in maneuvering the dam, every minute of delay is of importance to navigation. The reader should bear this condition in mind while reading the long argument (pages 1288 to 1292†) for more bear-traps and the entire elimination of weir wickets.

Mr. Williams gives an interesting record of the trap of highest lift of which the writer knows.

In reading Mr. Duis' reference to the Dam No. 18 bear-traps and weir, it should be remembered that the dams on the upper river were not built in order. When Dam 18 was started and when the bear-traps were designed and constructed, there was not a steel bear-trap in operation on the river, or in the world, approaching it in stiffness or weight per square foot of lower leaf. At that time only Dam No. 1, with only one 50-ft. wooden trap, had been placed in operation. The writer visited that dam in 1902, and the trap was out of service for repairs. It is the great improvement, in the No. 18 and the other steel traps with leaves of the same design, which has reduced the usage of the Chanoine wicket weir, notwithstanding the excellence of weirs of that type operated from a bridge, as long demonstrated by the eight dams on the Great Kanawha River. The six dams referred to on page 1288† are Dams Nos. 1 to 6, inclusive, on the Ohio River, all of which were commenced several years in advance of Dam No. 18. Wooden traps were first designed for several of them and for some of the other dams. The change to steel for Dams Nos. 2 to 5, inclusive, was decided at about the time the No. 18 plans were completed or after. The span of the steel traps for those dams was made 91 ft. to fit masonry which had been built before the decision as to building the steel traps, and thus that span has become the standard for the river. When Dam No. 18 was designed, the regulation of all the upper pools then in operation in the United States was being done principally or entirely by the use of wickets. That method had been used extensively in Europe for many years. The operation of wickets from a bridge is the only method of pool regulation on the Great Kanawha. Although the improved bear-trap is superior to the wicket as a regulating weir, under the direction of the builder of the Kanawha dams, their operation appeared to be excellent until the No. 18 type of trap was placed in commission. During the 5 years of operation of Dam No. 18, under the charge of the writer, the wicket weir was used as an important factor in regulation and operation. In later years, due to the ease of maneuvering the

* *Proceedings*, Am. Soc. C. E., January, 1922.

† *Proceedings*, Am. Soc. C. E., May, 1922.

bear-traps, the inclination appears to be to use the traps for that purpose to their limit before seeking assistance by use of the wickets. Although the improved bear-trap has reduced the use of the weir wickets, it seems somewhat premature to state that wickets are "obsolete". This seems especially true as long as those in authority continue to design and build new dams with wicket weirs, as indicated in the discussions by Mr. Thomas and Mr. Grimm. The writer regrets that they did not amplify more of the subjects introduced so briefly in the paper.

Besides the improved arrangement of wicket weirs in Dams 34, 36, and 38, described near the bottom of pages 1277 and 1279* it is gratifying also to note that, in some of the new plans, the filling and emptying of culverts for the bear-traps is being planned about as suggested in Paragraph (*c*), page 41.†

In referring to the formula, $Q = m\ (L'\ H')\ \sqrt{2\ g\ (h\ +\ Z)}$, page 14,† Mr. McAlpine pertinently says that "the lack of authentic experimental data as to the proper values of *m* makes the use of this formula unsatisfactory". The writer diligently sought a more applicable formula, and more data as to values of *m*, without satisfactory results. He doubts if any exist. It is believed that satisfactory data must be obtained from completed dams in operation, and not from models. In the particular case use on page 14,† from observations of the discharge through the completed dam, the writer believes that the actual discharge is quite close to that computed (57 700 cu. ft. per sec.), and he is confident that the discharge computed on page 1294 (33 900 cu. ft. per sec.), equivalent to the natural discharge of the river at a 7-ft. stage, is too small. The writer fully agrees with Mr. McAlpine that it is time that some of these formulas and data were tested by experiment, and that all desired values be obtained by careful observation and velocity measurements at all stages below the crest of dams.

The apparent error in the areas of discharge, referred to by him, is due to the fact that the gauge observations were made at a point about ½ mile below the dam, up to a stage of 10.7 ft., the highest stage at which observations were made. These areas, from a section at that place, were used up to the 11-ft. stage, whereas, above 11 ft., the areas were taken from the cross-section of the river on the axis of the dam.

Herman Haupt, a Civil Engineer, published, in 1855, an article on the Ohio River problems, improvement of navigation and flood control by use of restraining dams, in which he says, "the control of floods, therefore, may be left to future legislation. It will be sufficient at present to attend to the improvement of the nagivation. To accomplish this object will be glory enough for one generation".

Most members of this Society were born after these words were written, and have seen engineering inventions and works, greater, more numerous and more wonderful than those by all the preceding generations combined. They indicate a surprising vision for one of that day when the wonderful discoveries and improvements of this generation were hardly commencing.

* *Proceedings*, Am. Soc. C. E., May, 1922.

† *Proceedings*, Am. Soc. C. E., January, 1922.

It is a great pleasure to the writer to know that he has taken even a small part in the solution of the problems relating to the Ohio River project and improvement, one of the many great works conceived, designed, and executed by his fellow workers of this Society.

As a last word, the writer wishes to express sincere and grateful appreciation to each of the contributors for the valuable matter added to this paper, by their discussions.

THE WATER POWER PROBLEM

A SYMPOSIUM

Discussion*

By F. P. Williams, M. Am. Soc. C. E.

F. P. Williams,† M. Am. Soc. C. E. (by letter).‡—Considering the test that the electric power business has been passing through during the past few years, in the matter of taking care of both the increased demand and the maintaining of rates when everything has advanced in price, it is of especial interest to note in Mr. Hogan's paper§ the tendencies of water power development at the present time, and their possible effect on future conditions.

A study of a score of scattered public service companies operating in New York State during the period from 1910 to 1920, shows that the average revenue per kilowatt-hour for all classes of service, has remained substantially the same. It is believed that the companies are representative, and the showing noted is typical. This maintaining of rates has happened in the face of the rise in cost of practically everything since 1914.

An important factor that has helped to secure the record, especially in regard to rates, is the increase in the proportion of water to steam power. In 1910 the energy generated by companies reporting to the Public Service Commission, outside of New York City, amounted to a total of 950 000 000 kw-hr. In 1910 the hydro-electric energy generated was 4.4 times the amount of steam-generated energy; in 1920 the total generated was 3 200 000 000 kw-hr., or 3.4 times the amount generated in 1910. In 1920 the hydro-electric energy generated was 5.2 times as great as that generated by steam, indicating an increase of 0.8 in the ratio of hydro-electric over steam power. The present tendencies, pointed out by Mr. Hogan, indicate how additional power may be available through economies in practice, and by further development. The result will probably be an increase in the ratio of hydro-electric power to steam power. Although the conditions just noted have obtained in the operation of public service corporations, private steam plants have had to face more pronounced increased operating costs, especially of fuel. It is of still greater interest, therefore, to such power users that additional hydro-electric power be developed.

In view of the power conditions that have obtained during the past few years, it is very desirable that the tendencies outlined by Mr. Hogan be followed for the purpose of maintaining that record and improving it, if possible.

* Continued from March, 1923, *Proceedings*.

† Secy., New York Water Power Comm., Albany, N. Y.

‡ Received by the Secretary, February 23, 1923.

§ *Proceedings*, Am. Soc. C. E., November, 1922, p. 1741

Although the fact that the ratio of water power has kept pace with the steady, annual increase in the amount of power required, has increased over the steam power, and has been of great value in maintaining rates, there have been other economies helping toward the same end, such as improvements in machinery, greater efficiency in transmission, greater general refinements in practice, and changes in control, that have contributed to that end.

That an increased proportion of water power has not been the sole element tending to keep rates per kilowatt-hour from rising, is shown by the lowering of rates during the past few years in New York City, a steam-generating center. The average revenue per kilowatt-hour, as reported by the public service electric companies, except street railway companies of New York City, in 1908, was 6.9 cents. In 1919 the average revenue was 4.4 cents. This is all steam-generated energy. In 1919 the amount sold was 1 104 466 493 kw-hr. In the case of the city plants, the coal required to generate 1 kw-hr. in 1908 was reported to the Public Service Commission as 4.20 lb. In 1919 it was 2.38 lb., or 0.6 of the former quantity, showing a much greater economy in the use of coal.

Although a great deal has been accomplished in keeping down costs, by invention and refinements in operating, it does not appear as safe to rely on economies in this direction in the future, as to rely on following the present tendencies outlined by Mr. Hogan, especially in securing greater economies through the construction of storage reservoirs, the inter-connection of service, etc. For example, the present developed hydro-electric power may be trebled by the increase in power that may be secured in present plants through river regulation, and by the construction of plants to use the undeveloped power.

Storage reservoirs for regulating stream flow hold promise of great general benefits. Regulation will result in lessening flood damages, in increasing power output, and in improving navigation in the large streams by increasing the low-water flow. However, when it is suggested that storage may be carried to a point that will make Adirondack powers independent of steam, if desirable, it must not be overlooked that, in the case of this power, which requires to be transmitted a long distance to market, auxiliary power will always be desirable, if for no other reason than to protect against interruption of service, to which all long-distance lines are subjected.

As indicated, the conservation of the natural resources of New York State has been studied carefully, both by public and private authorities. Stream flow records, forming the scientific basis for any development, have been generally maintained. Although the power record of the past few years has been a noteworthy one, there appears to be no doubt that the future will give a good account of itself, if the tendencies outlined are advanced rapidly enough to meet conditions. Fortunate, indeed, is the State that has ample coal deposits. Happily, by the advancement made in hydro-electric art, and by the careful study and development of their water powers, other States may approximate the coal advantage of their sister States.

THE DESIGN OF STRUCTURAL SUPPORTS FOR TURBO-GENERATORS

Discussion*

By R. von Fabrice, Esq.

R. von Fabrice,† Esq.—The author has entered a field of civil engineering most intimately connected with the Mechanical and Electrical Professions. As a matter of fact, the subject treats of the execution of the design of a structure, and may mean either success or failure in the proper functioning of the prime mover of the modern power station. Therefore, it is recommended that the American Society of Civil Engineers should try to formulate specific recommendations covering the various phases of the design of structural supports for turbo-generators, which could be used as a basis in executing the designs of these important structures.

Structural steel supports, as foundations for prime movers, are of comparatively recent origin, the earlier installations having caused considerable difficulty, chiefly due to lack of knowledge pertaining to the specific characteristics of the equipment to be supported. This lack of proper information was not confined solely to the civil engineer, but was shared equally by mechanical and electrical engineers. It is not an infrequent occurrence, at present, for builders to be inconsistent in their instructions and data concerning the deflection allowable, the distribution of loading, and the rigidity of the component parts of the equipment.

Most of the manufacturers do not depend on the base-plates of their equipment for equal distribution of loading, and consequently the base-plate serves only for centering the component parts of the unit. This, however, is only true for those turbo-generators which have a continuous base-plate. Some of the builders of larger turbo-generators do not even provide a continuous base-plate for their unit, and thus depend solely on the foundation or support to hold the various parts in alignment, supplying an additional difficulty to be overcome by the civil engineer, and incidentally also greatly increasing the cost of the structural support.

The cost of structural supports for turbo-generators having well-designed, continuous, rigid and adequate bases, is approximately 25% less than the cost for the other and more commercial types.

The main difficulties encountered in the design of the structural supports are the space limitations on account of auxiliary equipment, the limitations

* Discussion of the paper by Edward H. Cameron, Assoc. M. Am. Soc. C. E., continued from March, 1923, *Proceedings*.

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of allowable deflection in the structure, and inadequate provisions for the supporting or bearing surfaces on parts of the turbo-generators themselves.

The three types of pedestals mentioned by Mr. Cameron, namely, reinforced concrete, structural steel, and combination structural steel and concrete, or composite type, are not adaptable to all kinds of installations, and the proper selection of the type to be used for any specific case is dependent on the auxiliary equipment, type of unit, space conditions, and arrangement of equipment.

The structural steel type has the greatest advantages, as it provides the most accessible arrangement for the auxiliaries and piping, and, further, possesses the greatest positive mathematical determination of design.

The composite type consists of a structural steel pedestal entirely encased with heavy concrete. The main practical advantage is that the encasement of concrete provides volume or mass, and incidentally a protection to the structural steel against temperature changes and consequent distortions of the structural frame. This last item, however, can also be obtained by a 2 or 3-in. thickness of concrete over the structural steel members, and this protection is of greatest value for the columns. The structural value of the heavy concrete encasement is uncertain, and it is advisable to ignore the structural value of the concrete in the calculations for strength and rigidity. Therefore, it would seem advisable to encase the steel only against temperature distortion, and design the steelwork as a purely structural support, of adequate strength for deflection and vibration.

Sudden and local temperature changes in the turbine room basement will distort the structural type of pedestal sufficiently to cause serious disturbance in the turbo-generator itself. Therefore, some engineers believe it advisable to protect the columns against such temperature changes. In any event, such protection can be obtained at a comparatively small additional cost.

The reinforced concrete pedestal is becoming less popular in modern installations, and is unsuitable for a compact and accessible arrangement of auxiliaries. Furthermore, it is impossible, in most instances, to obtain the structural strength in cross-girders, as required by the limitations of deflections imposed by the manufacturers and, also, by the inadequate bearing surfaces provided on the equipment itself at the points of most critical concentration of loading. However, this is not the case for units when continuous, heavy and properly designed base-plates are provided by the manufacturers.

Vibration in the pedestal can be reduced to the minimum in any type of pedestal, and is dependent on the proper methods of bracing, design of connections, proper detailing of connections, and primarily on the proper balancing and adjusting of the unit itself.

To provide properly against vibration in pedestals of the structural type, it is imperative that bracing in all directions be supplied, and that the columns be secured to the grillages and girders with heavy brackets and knee-braces.

It is advisable to cross-tie the grillages so as to have at least one beam of the grillage continuous under the entire set of supporting columns in both directions, and to provide heavy brackets riveted to the column and grillage.

This will stiffen the entire structure and guard against looseness in the connections between the columns and grillage. Further, all tiers of the grillage should be riveted together, and the column base should be bolted or riveted securely to the grillage.

All tension and compression members should be fabricated to provide initial tension or compression, and in practice it has been found advisable to fabricate tension members $\frac{1}{16}$ in. short in 10 ft. and compression members $\frac{1}{16}$ in. long in 10 ft. This will eliminate all possibility of slackness in the joints. It is also advisable to increase connections for all abutting members by at least 50% of the theoretical connection value, in order to provide rigidity at the joint itself and eliminate loosening of connections later. This further tends to throw the oscillations or vibrations into the members themselves, rather than into the joints or connections.

All girders composing the platform of the pedestal supporting the turbo-generator should be designed for deflection only, and the summation of all deflections of all abutting or connecting girders should result in a uniform deflection under all pedestals or bearings of the turbo-generator along its longitudinal center line.

The top of the steelwork, for a structural support, should be arranged so that at least 4 in. of concrete and grout are possible between the base of the turbo-generator and the steelwork to insure against pulverization of the grout after a period of operation.

The space between the girders of the platform should be filled with concrete in order to provide a massive mat to absorb the pounding of the unit and add to the rigidity of the joints. All parallel supporting girders under any supports of the turbo-generator should be tied together by heavy plate and angle diaphragms. Pipe and rod or standard cast-iron separators are not reliable for this purpose.

Adequate provisions should be made for foundation bolts holding the turbo-generator to the pedestal, and in cases where supporting girders come in line with holding-down bolts, a specially shaped billet, tapped for the holding-down bolt, is to be securely riveted to the top of the girder.

In the design of structural steel pedestals it is not safe to base the design on any fixed unit fiber stress as a limiting feature governing the size of the girders. Each girder should be designed purely for the proper required deflection for the load to be supported. The designer must make proper allowances for impact and distribution of loadings or weight of the unit on the various girders of the platform.

For the design of girders, three times the actual net weight of the unit has been found to be ample to provide for impact and dead weight of the structure itself. For the design of columns and grillages, three times the actual weight of the unit, together with the weight of the platform, should be used. Grillages spanning a canal, or not being supported uniformly, should also be designed for deflection, and this deflection should be considered in the design of the platform and entire structure.

Continuous girders parallel with the longitudinal axis of the turbo-generator should be designed as such, and all positive and negative moments

should be considered. Ample stiffness should be provided at all cross-connections and over columns. Turbo-generators having a substantial, well designed, and self-supporting base-plate do not require continuous side girders, and consequently a less expensive pedestal can be surely designed, which can also consist of two separate structures, if so desired, supporting separate portions of the unit.

The entire structure should be supported on a continuous heavy concrete mat of adequate thickness to guard against shear and to avoid local settlement under any individual column. Steel reinforcement is required when the volume of concrete available is of limited thickness.

When setting the turbo-generator on the pedestal, it should be wedged securely to the proper alignment with steel wedges. The unit should then be grouted in place with a rich cement grout which should be poured so as to fill practically the entire hollow portion of the base-plate, as this will provide for shrinkage as the grout dries and hardens under the unit. The steel wedges should not be removed, but cut off, when they extend into openings required for air ducts, piping, or other auxiliary parts.

When installations are made, care should be taken to avoid any rigid connections between the structural support of adjacent units, building wall, and building steelwork. This is primarily important in the direction at right angles with the longitudinal axis of the unit. All galleries around the turbo-generator, and secured to the pedestal structure, should have all grating and cover-plates (if any are used) bolted securely to the steelwork, in order to avoid unnecessary disturbances. If concrete floor-slabs are supported on these galleries, they should be haunched on the lower flanges of the supporting beams, to insure against loosening from the supporting steel and consequent disturbances.

The method of supporting the condensers depends largely on the size of the unit and the design of the condenser and turbine.

When an expansion joint of adequate design is used, the condenser can be supported either on pedestals from the basement floor, on steelwork connecting to the columns of the pedestal, or from hangers from the supporting platform itself. The latter method is not to be recommended, as it adds undue loading on the girders supporting the turbo-generator, and consequently increases greatly the cost of these members, on account of the small allowable deflections in these girders.

For large installations, the method of supporting the condensers on properly designed and adjustable spring supports is the most satisfactory, especially if there are provisions for testing the actual distribution of the condenser weight by suitable hydraulic jacking devices. With this arrangement, positively accurate distribution of loading due to the condenser is possible, and absolute assurance of the proper amount of dead weight required by the builders on the casing of the turbine due to the condenser is insured. Also, blocking devices should be provided to furnish proper means for supporting the condenser when filled with water for testing purposes to guard against undue strains on the turbine casing under these conditions.

For new turbine-room designs, the designers should allow ample head-room to provide space for all auxiliaries, to avoid the necessity of providing pits under the equipment for condensate pumps, and other auxiliaries. The following figures will provide sufficient head-room between the basement floor and top of the turbine foundation for the various units:

For	units	from	30 000	kv-a.	to	45 000	kv-a.,	or	larger:	35	ft.
"	"	"	20 000	"	"	30 000	"	"	"	30	"
"	"	"	12 000	"	"	20 000	"	"	"	25	"
"	"	"	5 000	"	"	12 000	"	"	"	20	"

apers.

head-
iding
The
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REINFORCED CONCRETE COLUMNS

Discussion*

By Messrs. Edward Godfrey, E. S. Martin, George Paaswell, Jacob Feld, and A. W. Buel.

Edward Godfrey,† M. Am. Soc. C. E. (by letter).‡—The synopsis of this paper holds out the hope that the author will perform a needed service in presenting sound reason for revising the Joint Committee Report on matters relating to columns. After reading practically all of Mr. Tucker's paper, the writer is not convinced that designing would be on any better basis if his recommendations were adopted than it is on the basis of the Joint Committee Report.

The Joint Committee Report of 1909 allowed 450 lb. per sq. in. on concrete in a rodded column, but excluded a protecting shell, 1½ in. thick, the concrete in a 10-in. column (2 000-lb. concrete) being safe for 22 050 lb.

The Report of 1912 allowed 450 lb. per sq. in., and excluded a 2-in. shell, the same column being safe for 16 200 lb. on the concrete.

The 1917 Report allowed 450 lb. per sq. in., but did not exclude the shell, the same column being safe for 45 000 lb., or twice what it was safe for under the 1909 Report and nearly three times what it was safe for under the 1912 Report. With exactly the same data on which to base judgment, totally different standards were evolved.

The first two Reports of the Joint Committee permitted the use of plain concrete columns up to twelve diameters. These same reports indicated clearly that the function of ties in rodded columns was merely to hold the upright rods in place while the concrete was hardening. Any kind of thin wires would answer this purpose. The later reports specified rods of a certain size for ties, and a spacing not to exceed sixteen diameters of the longitudinal steel.

These facts, together with a clearly-defined dissent from the standards of the Joint Committee, referred to by the author on page 180,§ suggest the need of an engineering commission to investigate the subject of columns. This commission should have open and public proceedings, should listen to any one wishing to be heard, and should render specific answers to arguments against the standard.

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* This discussion (of the paper by John Tucker, Jr., Esq., published in February, 1923, *Proceedings*, and presented at the meeting of February 14, 1923), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Structural Engr. (Robert W. Hunt & Co.), Pittsburgh, Pa.

‡ Received by the Secretary, February 10, 1923.

§ *Proceedings*, Am. Soc. C. E., February, 1923.

It is difficult to understand the method by which the author undertakes to prove his case against existing standards and sustain his substitution therefor. There are parts of the paper that seem to contradict other parts: The author states, on page 180:* "Even those engineers who most favor this type of column [the rodded column] cannot agree on the value of the rods nor can they prove that they have a definite value." Then, on page 207,* he states: "Longitudinal reinforcing rods add definite strength to the column."

On page 231,* in the criticisms of the "1921 Tentative Specifications", the author states:

"The working stresses, in all cases, are lower than the safe calculated stresses, provided the concrete in the column is equal to the nominal concrete strength on which design has been based."

In the next paragraph he states:

"For the strengths to be expected from a nominal strength concrete the permissible stresses of the 1921 Tentative Specifications are much above the safe calculated stresses for many possible selections of variables."

Surely working stresses and permissible stresses are the same; but how can such stresses be lower than the author's "safe calculated stresses" in all cases and much above the safe calculated stresses in many possible cases.

On page 190* he states that it is impossible to determine the true stress in the rods; and later, in the same paragraph: "It will be shown subsequently that the rods develop a very constant strength."

It is stated on page 220* that: "A unit volume of concrete has the same ultimate strength no matter where it is placed." On page 187* a formula is given for the "increase in strength of concrete due to spiral reinforcement", thus proving that if the volume of concrete is placed inside a spiral it will be of much greater strength. The author states on page 189* that "the spiral increases the strength of the concrete in a definite manner." Surrounded by restraining concrete, and carrying a bearing load, a cubic inch of concrete has many times the compressive strength of a free cube; so that a cubic inch of concrete, wherever placed, has by no means a certain and fixed value as a resistance to compressive stress.

On page 227* the author says:

"The contention is untrue that, in the process of destruction of the [rodded] column, the rods buckle and cause the surrounding outer layer of concrete to scale, thus starting and aiding the failure of the column. The phenomenon is a phase in the failure of the testing of this type of column, and represents the buckling of the rods after the maximum strength of the column and concrete has been passed and the concrete has broken, allowing the rods to buckle outward."

Apparently, his conclusion here is based on the suddenness with which the whole column breaks down. On page 209* he refers to the suddenness with which the Quebec Bridge collapsed. Would he say that the entire Quebec Bridge reached its ultimate strength in every part and that no one part failed first, thus causing the whole collapse?

* *Proceedings*, Am. Soc. C. E., February, 1923.

In addition to the foregoing contradictions, there are statements and attitudes toward engineering problems that are difficult to harmonize with fact and logic.

On page 218* he states: "Increase in the factor of safety, a result of including too many and extraneous factors, does not give increased safety," and again, in the same paragraph, "The smaller the factor of safety the safer a structure becomes." Is it the actual dimensions and type of design that give stability to a structure, or the correctness of the logic of the designer—the idea in his mind?

The author's statement about reliability is difficult to connect with any well-ordered set of facts. When reliability is mentioned, one thinks of something dependable, not likely to fail suddenly and without warning; but the author rates high in the scale of reliability the very things that are the most treacherous. He states, on page 209.*

"The suddenness and violence of the failure or the length of time required for failure to occur in the testing machine is no indication of the strength reliability of the material or structural element."

On page 233* he states: "The rod and spiral reinforced concrete column is shown to be almost as reliable as the rodded column." This sounds like irony. Hundreds of rodded columns have failed in the general collapse of more than a score of structures; and no structure having spirally reinforced columns has collapsed. The author states, on page 232,* that the spirally reinforced column is less reliable than the plain concrete column. Then why use reinforcement in columns?

On page 173* the author states, under "Unreinforced Columns": "Test results of only eleven such columns can be found." Then he gives twelve, and, among these, he selects seven out of nine made by Messrs. McKibben and Merrill. Professor Talbot, in *Bulletin No. 20,* reports tests on 19 plain concrete columns, and other experimenters have made tests on such columns. Why this meager selection of data, when an attempt is made to establish a principle?

On page 183* the author states that the rods in a column "have a strength of 39 700 lb. per sq. in. for a column of infinitely small length". A vertical rod, even of finite length, will have an ultimate strength that no testing machine could measure. Throughout the paper, the author ignores this vital fact, that thin disks of any material are enormously stronger than cubes or blocks of ordinary dimensions. It is incorrect, therefore, to mention the strength of any column of infinitely short length.

The whole tenor of the paper seems to be an effort to predict the laboratory result when columns are made or tested in the laboratory or under the most perfect conditions. Is the laboratory used merely for the purpose of discovering and predicting what results may be expected in any given series of laboratory experiments subsequently made; or, is it a tool of the designing engineer; and are results to be interpreted with a view of helping him to solve the problems of stability? In the laboratory, columns are tested with a balanced

* *Proceedings,* Am. Soc. C. E., February, 1923.

or symmetrical load. In practice, a column with a balanced or symmetrical load is exceedingly rare, as rare as a fixed-ended column; and this, again, is practically the only kind of column tested in the laboratory.

In the author's mind, reliability seems to be the probability that a given column will develop a certain predicted strength in a laboratory test, and no more. (He rejects some spiral column tests because the results are too high.) In the engineer's mind, reliability is something to be depended on, even if materials and manipulation in the field are not perfect. The author finds most reliable the things that many engineers have found to be least reliable. The rodded column, which he classes high in the scale of reliability, has been the subject of more failures than any other structural element.

Toughness, the quality of withstanding overload without distress, of taking unequal stress, either because of unbalanced loading or of inequality of the concrete in different parts of the column—this quality has been the saving of many a structure none too well designed. Its opposite, brittleness, has caused the failure of a large number of structures. The author, however, in his scale of reliability, rates the most brittle columns higher than the toughest.

It is futile to say that one must eliminate irregularities of loading or irregularities in the making of concrete columns. One might as well try to control the weather as to introduce laboratory methods on the job, and, in fact, the weather has a great effect on the quality of the concrete.

The author ridicules the common use of a factor of safety, and attempts to show that it is not based on scientific principles. The phrases, "scientific basis of engineering knowledge", "rigid control of the making and placing of concrete", "advance in technique", sound well; but, if there is no margin to cover uncertainties in execution and unknowables in design, what is to be done, if a batch of bad concrete is placed in a rodded column, or some concrete is affected by frost? The engineer who does not exert himself to make his structures fool proof, not only by using a factor of safety to cover contingencies but also by using a design that will produce tough columns, is likely to experience serious failures.

Important as is the factor of safety, to insure a substantial structure and to make every part capable of taking its load safely, it is yet more important to care for the general stability of a structure by adopting a proper design, members with proper reliability, and a structure in which, on the whole, no principle of equilibrium is violated. Factors of safety did not enter into either of the failures of the Quebec Bridge.

Reinforced concrete failures, contrary to the statement on page 221*, have not been due to inferior concrete, except where such concrete may have started the failure, and brought on the collapse of a brittle and unreliable structure. In these structures an increased factor of safety, exhibited by merely adding more brittle concrete to improperly designed columns, would have had little if any benefit.

If laboratory tests, instead of being made on isolated members, ideally constructed, and conditioned, were made on a pair or on four columns, monolithic

* *Proceedings,* Am. Soc. C. E., February, 1923.

with girders, the inherent brittleness and unreliability of the rodded column would have been manifested long ago. One test was made in this way, and the columns failed at about 400 lb. per sq. in. Failures have shown that the stresses in rodded columns were as low as 150 lb. per sq. in.

It will doubtless be argued that the failures of columns at low unit stresses are due to eccentric loads. It is easy for the experimenter and specification writer to say that eccentric loading should be taken into account in the design, but it cannot be done. The writer does not insist on it, except where the eccentricity is obvious, as where the plane of a girder is to one side of the column axis, or where a steel girder rests on a bracket and has no web connection to the column.

Given the simplest case, of a steel girder supported by two columns to which it is riveted by a web connection: A practical designer will design the girder for a span equal to the distance from center to center of the columns, and will design the columns for the direct load. He will be right, and his structure will carry its designed load; but the theoretical designer will doubtless compute a bending moment in each column, and design the girder for the shortened span. The former method is the way designing is done, and probably always will be done, in spite of all the books and specifications that may be written.

Suppose these same parts are to be designed in reinforced concrete: If the columns are hooped, the same method of computing the bending moment in the girder and the direct load in the columns may be used with safety. If the columns are of the rodded, brittle, unreliable type, the designer who does the same thing does it at his peril, especially if the girders are long or shallow. The latest large failure of record, that at Salina, Kans., is an example.

The author excuses all low values in column tests, and explains all high values on the convenient basis of the great variation of the strength of concrete. By thus allocating any desired value to the strength of the concrete, of course, a reinforcing value of very definite amount can be found for upright rods, no matter what strength the column may show in the test. Among Professor Talbot's tests, in *Bulletin No. 10,* one plain concrete column stood nearly 50% more load than one rodded column; and, if four tests had not been made, the average of the plain columns would far exceed that of the rodded so-called reinforced concrete columns. If the author can make out of this an argument that upright rods reinforce a column, he is welcome to any assurance it may give him for his laboratory work; but he should not ask designers to risk valuable lives and property on this kind of assurance.

Of the piers tested by the Bureau of Standards,* the hooped piers having 1% of longitudinal steel are, on the average, only $1\frac{1}{2}$% stronger than those having no longitudinal steel, instead of 15% according to standard rules. One hooped specimen with no longitudinal steel is stronger than any of those with 1% longitudinal steel, and is exceeded by only one of the group with 2% longitudinal steel. One hooped specimen with 6% longitudinal steel is only 10% stronger than a specimen with no longitudinal steel, instead of 90 per cent.

* Published in *Proceedings,* Am. Concrete Institute, February, 1915.

Of the rodded piers, the group with 2% longitudinal steel is weaker than that with 1 per cent. There is no doubt that, if these specimens had been columns instead of piers, the results would have been more erratic. If these facts demonstrate anything, it is, to quote the author, that engineers who favor the rodded column cannot prove that the rods have any definite value. Attempts to harmonize such erratic results by arbitrary shifting of concrete values should be anything but convincing, especially to men who are responsible for the safety of structures.

In dealing with spiral reinforced concrete columns, the author fails to recognize the one thing that explains the added compressive strength which he finds to exist in the concrete. He even omits the data by which this added strength might be judged or duplicated. The thing referred to is the pitch of the coils. It is not sufficient to say that a column is reinforced with a coil, nor to give the area of the coil. The pitch of the coil is of great importance. In a hooped column, the succeeding hoops make of the column a system of superimposed flat disks. It is well known that a flat disk of any material, such as concrete, is of very much greater strength than a cube or a cylinder two diameters in height. A cube of lime mortar will stand very little compression, but a thin joint of the same mortar will stand a heavy pressure, and the thinner the joint the greater its carrying capacity. This effect is independent of the lateral confinement, except the confinement that is afforded by the friction of the end planes. In the hooped column, the hoops offer a corresponding confinement. In the spiral reinforced column, a somewhat similar action takes place, and the pitch of the spiral is of great importance. In the tests shown on page 185*, nearly all the test columns have about 1 in. pitch in the spiral. The deductions made by the author, therefore, are of limited application. Columns with a different pitch in the spiral might show very different strength.

When groups of tests have to be excluded to make a formula come out right, it does not speak well for the formula. If these groups were included, it would be seen that the percentage of spiral reinforcement has little to do with the added strength of the concrete. Some columns with one-half of 1% spiral reinforcement have a greater increase in strength than others in which the reinforcement is only 1 per cent.

Spiral reinforcement corresponds to lattice in steel columns. Steel columns would be deficient, if the lattice system was inadequate, but the area of the lattice bars is never included in the area of a steel column, nor is the calculated strength of the column varied with the kind of lattice. The lattice, however, must be sufficient.

Thus there is doubtless a balanced proportion where the coil area and pitch are in conformity, and it is improper to attribute strength to a column varying with the coil area. If experimenters would endeavor to discover the proper balance between the coil area and the pitch to produce a well-proportioned column, it would be of great benefit to the Profession.

The old, discarded formula of Considère and the author's substitute are both wrong, because they would vary the strength of the column with the vary-

* *Proceedings*, Am. Soc. C. E., February, 1923.

ing area of the hooping, when there is nothing in the compression area between the hoops but concrete (and the upright steel, when it is used); and the nature of these is not altered by adding to the steel area. Like the lattice in a steel column, the coil area should be sufficient.

The proper way to proportion a column is to use a unit stress well within what a plain concrete column would stand under a central load, as this, by many tests, is proved to be the point where the column will begin to fail, no matter how it is reinforced. It is the outer shell that governs in the matter of unit stress.

The author belittles the contention that shrinkage of the concrete in a rodded column causes high initial compression in the rods, which buckle out and spall the concrete, thus causing the failure of the column. He states that the rods do not buckle until the whole column has reached its ultimate strength; but, how can he prove this? He cannot deny that in many tests the rods buckle and push out the concrete. Simultaneous with this action, in a heavily loaded column, the center of the column would have to crush. The author will have to exhibit some tests in which the columns broke by splitting and the rods were left embedded in the concrete and straight, if he wishes to satisfy anybody that the rods do not buckle and push the surface concrete out, and thus cause the failure, in these cases. In the Edison Building, many rods buckled out and pushed off the concrete, and the columns were still standing and carrying their load. Some of these rods were in the middle of the side of the column. This happened even in rooms where the heat was not sufficient to melt insulation on electric wires.

It is far from convincing to mask all irregularities and account for every freak result by the blanket statement that the concrete of the column is of irregular and unknowable strength, as the author has done.

The von Emperger type of column, with its cast-iron core of special, high-strength, cast iron, and its hooped concrete shell, would better be left as a laboratory curiosity. There is no practical way to make a good splice in the cast-iron core, either for the high compression or for bending, to impart toughness to the system. Shop milling is not sufficiently accurate for this purpose. Flanges would break the continuity of the hooped shell. Besides, it is difficult to connect beams and girders to columns of this type.

On page 232*, the author states: It is immaterial how a structural element fails", and on page 222 he states:

"It is immaterial * * * whether the rods of a longitudinally reinforced concrete column cause the failure of the column or whether the failure is independent of the action of the rods."

On the same page he intimates that the ground for this non-concern is because the same phenomena occur in the test columns. The engineer is vitally concerned with what makes a structural element fail. In no other way can he avoid the recurrence. Suppose, for example, that it were established that the rods in a rodded column did cause the failure by buckling, the simple expedient of using close-spaced hooping would prevent failures of this type.

* *Proceedings*, Am. Soc. C. E., February, 1923.

E. S. Martin,* Assoc. M. Am. Soc. C. E. (by letter).†—Although Mr. Tucker's paper presents the most thorough study of reinforced concrete columns that has come to the writer's notice, he will discuss only one point at this time.

It is recommended that unit stresses be reduced 40% if field cylinder tests are not made. This is an important point affecting very materially the size and costs of columns, and yet apparently this recommendation is not supported by anything in the paper.

Of what value are field cylinder tests? It is the usual practice to cast two or three pairs of cylinders from a day's run consisting of 200 to 400 batches. In testing open-hearth steel at the mill the engineer requires tests to be made of each charge or heat, and would not regard very highly a report of tests made from one heat out of a hundred, although that is all a field cylinder test of concrete is.

In the writer's judgment, field cylinder tests are indications only of the quality of the concrete that can be made from a particular combination of aggregates and cement; and it is only necessary to make these tests for fresh supplies of either.

George Paaswell,‡ M. Am. Soc. C. E.—This paper makes no pretense of being a contribution to column literature *per se,* but it is unquestionably a most stimulating contribution to reinforced concrete, from the standpoints of both design and experiment. As the reinforced column is by definition a compression member, it is the usual practice to design it by the accepted column formula, introducing such coefficients as experiment indicates, and making the formula, of course, a function of the slenderness ratio. As a matter of fact, it is seen by a study of the stress-strain diagrams (and it has been pointed out before) that, with the slenderness ratio of the usual concrete column practice, the column strength is not a function of $\frac{L}{D}$. This leads the speaker to suggest the abandonment of attempts to mould the design formula to fit the column theory, and that a frank attempt be made to design the member as a heterogeneous block subjected to a principal compression stress, its failure being due to shear, internal friction, or otherwise, along experimentally determinable surfaces. The speaker suggests this method, in spite of the apparent complexity introduced into column design, with the hope that some light may be thrown on the, at present, hopeless muddle of spirally reinforced columns.

It is a matter of no great mathematical hardship to start from the comparatively simple elastic equations of isotropic solids and derive design equations. It is, however, a matter of grave decision as to what modifications must be introduced into the formulas to have them meet the variations of the given material from the ideal properties of an isotropic solid. To take a spirally reinforced column as an example of an isotropic solid makes extremely approximate the resulting equations of elastic action. This is merely by way of

* Secy.-Treas., James A. Wickett, Ltd., Toronto, Ont., Canada.

† Received by the Secretary, February 26th, 1923.

‡ Section Engr., Public Service Comm., First Dist., New York City.

gentle criticism, however. One must start with the isotropic conditions and then modify the analysis to meet the material variations.

The author would do well to emphasize the qualitative worth of the analysis rather than the quantitative. The Poisson ratio always breeds confusion and attendant distrust of analysis dependent on its determination. The ratio, presumably, is restricted to values between $\frac{1}{4}$ and $\frac{1}{2}$. As a matter of fact, there is no direct way of obtaining, experimentally, the value of the ratio. It can only be obtained by indirect methods. This is probably the reason for the great variation in its determination. Thus, Mr. A. E. H. Love* notes that Professor Voigt, in getting the elastic properties of certain crystals, having axes of isotropic action which are well defined, finds a value of the Poisson ratio of $-\frac{1}{7}$. The famous ratio is certainly a double-edged sword, and designers do well to hang very little analysis on it.

A Japanese investigator, Chido Sunatani,† has pointed out that the mechanics of failure are but little understood, and, as a result of experiments, has indicated a method of design predicated on a combination of shearing and internal friction action. Without vouching for the accuracy of these equations, aside from noting that the phenomena of failure are consistent with the investigator's equations, it seems to the speaker that more success could be obtained in either testing or mathematically analyzing the reinforced column along such lines, than in making a fetich of the column formula.

The matter of reliability is so ably and concisely put, and its application is so obvious, that one wonders that it has not appeared in print before. One cannot emphasize sufficiently the Society's debt to Mr. Tucker for his keen analysis of the application of statistical methods to the study of tests. It is true that an average test strength is meaningless until one knows the variations from this mean strength.

In the matter of working stresses, it must be always borne in mind that the concrete receives very little sympathy from the average contractor. In matters of decision it is the plant and not the concrete that receives the benefit of the doubt. On the other hand, a cylinder test is not usually a fair and typical indication of the strength and carrying capacity of the concrete structure. The building contains no structural unit stressed like the cylinder. Briefly, the cylinder test is not an absolute measure of the value of the concrete, but merely a relative one. The speaker hopes to read, some day, of extensive tests made on structural frames, in which the units are loaded in accordance with engineering practice, and not in accordance with the highly ideal conditions of "laboratory" columns and beams.

Jacob Feld,‡ Jun. Am. Soc. C. E.—There are two points in this paper which the speaker believes are not substantiated by experiment: The reduction formulas for column length, and the action and failure of spirally reinforced columns. The concrete columns used in practice seldom have an $\frac{L}{D}$ ratio

* "Treatise on the Mathematical Theory of Elasticity", 1st Edition, Vol. I, p. 96.

† The Technology Reports of the Tohoku Imperial University, Sendai, Japan.

‡ Brooklyn, N. Y.

greater than 15. The following tests (including those quoted by the author) of plain concrete columns show that the $\frac{L}{D}$ ratio has practically no effect on the allowable unit load, within the usual maximum $\frac{L}{D}$ ratio. It is not fair to compare the strength of a test column with that of a single test cylinder. It is better to compare the strength of columns with the average strength of the concrete used, as determined by the average of all the cylinders tested. The argument in favor of the method used by the author, that cylinder and column come from the same batch of mix, does not carry as much weight as it would in testing steel columns, comparing the column with a test specimen made from the same ingot. Such uniformity as is obtained in steel manufacture cannot be expected in concrete, where five or six men using equipment costing about $100 may constitute the entire construction as well as production unit. Comparison with the product of the steel mill is impossible.

In considering the M columns in Table 1,* the low values for the columns with a slenderness ratio of 16.6 are probably due to the exceptionally high test cylinder for that case. A comparison with the average cylinder values, including the tests at the University of Texas,† made by Professor F. E. Giesecke, M. Am. Soc. C. E., in 1919-20, at the University of Illinois,‡ at the University of Wisconsin,§ and the Aberthaw tests at the Watertown Arsenal (1897), is shown in Table 14.

TABLE 14.—EFFECT OF SLENDERNESS RATIO. PLAIN CONCRETE COLUMNS.

Source.	Diameter, in inches.	$\frac{L}{D}$	Ultimate Stress, in Pounds per Square Inch.				Ratio:
			Cylinder.	Cube.	Column.		
M..........	14	4.2	2 032	.....	2 070		1.02 to cylinder.
	14	8.5	2 032	.....	1 767		0.87 " "
	14	16.6	2 032	.....	1 747		0.86 " "
W_{466}.......	10.5	9.72	2 417	.....	2 600		1.08 " "
A. C. I., 1915	20.9	7.0	2 647	.....	2 745		1.10 " "
Illin. *10*, *20*.	12	10.0	.....	2 200	.....	(2 mos.)	0.78 to cube, 2 cols.
						(6 mos.)	0.90 to " 6 "
W_{800}.......	12 by 12	10.0	.....	2 300	.....		0.88 to " 4 "
Texas......	3 by 6	5 to 25	(5 sets of 30 columns each, wet mix.)				Little variation.
Aberthaw..	12 by 12	2 to 14	(hand-mixed concrete.)				1.00 ± 0.076.

Before drawing any conclusions from Table 14, the following notes, from the reports of these tests, should be considered:

"It is noticeable that there is but very slight, if any, decrease in strength with the length of the specimen, and this difference may probably be due to the fact that the longer columns were tested at earlier periods."

* *Proceedings*, Am. Soc. C. E., February, 1923, p. 173.

† *Engineering News-Record*, v. 90 (1923), pp. 274-276.

‡ *Bulletins 10* and *20*.

§ *Bulletin 300*.

The longest columns were tested in 35 days, the shortest in 47 days.*

"Strength of these concrete columns varied only slightly with the slenderness ratio, which, however, did not exceed 25; but it also shows that a variation in the consistency of the mix or a variation in the method of the placing of concrete may produce a very great variation in strength, more than 100 per cent." * * * "The strength of the column is almost independent of the slenderness ratio."†

"In considering these tests it must be borne in mind that the specimens [columns] were very carefully made."‡

Remembering, also, that the unit strength of cubes is greater than that of cylinders, there is no doubt that every one of these tests shows that the unit strength of columns is practically independent of the $\frac{L}{D}$ ratio. The greatest reduction is 0.14 for the M column with $\frac{L}{D} = 16.6$, the other tests show that $\frac{L}{D} = 8.5$ gave an unusually low value. A comparison of the author's reduction formulas with those of various specifications may be of interest, and is shown in Table 15.

A.—French regulations (1906): $P' = \dfrac{P}{1 + \dfrac{k}{10\,000}\,\dfrac{L^2}{r^2}}$.

*A*1.—For round-ended columns, k is 1.0; no reduction for $\frac{L}{D} = 20$.

*A*2.—For fixed columns, k is 0.25; no reduction for $\frac{L}{D} = 20$.

B.—German regulations (1905-07): for $\frac{L}{D}$ greater than 18, and a factor of safety, $s = 10$, $P' = \dfrac{\pi^2 E I}{s L^2}$.

C.—English (1909): $\frac{L'}{D}$: 15 18 21 24 27 30

$\frac{P'}{P}$: 1.0 0.8 0.6 0.4 0.2 0.0

C 1.—For round-ended columns, $L' = L$, the length of the column.
C 2.—For fixed columns, $L' = 4\,L$.

D.—Swiss (1909): for $\frac{L}{D}$ greater than 20, $P' = \dfrac{45}{35} \cdot \dfrac{P}{1 + \left(\dfrac{L}{100\,r}\right)^2}$.

E.—Austrian (1911): for $\frac{L}{D}$ greater than 30, $P' = P\left(1.72 - 0.024\,\dfrac{L}{D}\right)$.

F.—Los Angeles (1915): for $\frac{L}{D}$ greater than 15, $P' = P\left(1.6 - 0.4\,\dfrac{L}{D}\right)$.

* Aberthaw tests at the Watertown Arsenal, 1897.
† *Engineering News-Record*, v. 90 (1923), pp. 274-276.
‡ Univ. of Wisconsin, *Bulletin 466*, p. 14.

G.—Joint Committee, Tentative Specifications (1921): for $\frac{L}{D}$ greater than 20, $P' = P\left(1.33 - 0.033\frac{L}{D}\right)$.

H 1.—Tucker's formula for plain concrete columns: $P' = P\left(1 - 0.171\frac{L}{D}\right)$

H 2.—Tucker's formula for columns with longitudinal steel only:

$$P' = P\left(1 - 0.0265\frac{L}{D}\right).$$

H 3.—Tucker's formula for columns with rods and spirals:

$$P' = P\left(1 - 0.0183\frac{L}{D}\right).$$

Therefore, it seems that the author's recommendations for reductions in the unit loads, in order to allow for the $\frac{L}{D}$ ratio, are unduly harsh. The speaker was also impressed by the lack of consistency in Table 9.* Certainly there is no reason for allowing a less reduction for plain concrete columns than for the reinforced columns, as such reduction is meant to allow for the tension at the outermost fiber caused by flexure.

TABLE 15.—RATIO OF COLUMN STRENGTH TO CYLINDER.

$\frac{L}{D}$	*A* 1.	*A* 2.	*B*.	*C* 1.	*C* 2.	*D*.	*E*.	*F*.	*G*.	*H* 1.	*H* 2.	*H* 3.
10	.....	.....	....	....	....	....	....	....	....	0.829	0.735	0.817
11	.....	.....	....	....	....	....	....	....	....	0.812	0.708	0.809
12	.....	.....	....	....	....	....	....	....	....	0.795	0.692	0.780
13	.....	.....	....	....	....	....	....	....	....	0.778	0.655	0.762
14	.....	.....	....	....	....	....	....	....	....	0.761	0.629	0.744
15	.....	.....	....	1.0	....	....	....	1.0	....	0.743	0.602	0.725
16	.....	.....	....	0.93	....	....	....	0.96	....	0.726	0.576	0.707
17	.....	.....	....	0.87	....	....	....	0.92	....	0.709	0.549	0.689
18	.....	.....	1.0	0.80	....	....	....	0.88	....	0.692	0.523	0.671
19	.....	.....	1.90	0.73	....	....	....	0.84	....	0.675	0.496	0.652
20	.....	.....	0.81	0.67	....	....	....	0.80	....	0.658	0.470	0.634
21	0.586	0.850	0.74	0.60	....	....	....	0.76	0.63	0.641	0.443	0.616
22	0.563	0.838	0.67	0.53	....	....	....	0.72	0.60	0.624	0.417	0.598
23	0.542	0.825	0.60	0.41	....	....	....	0.68	0.56	0.607	0.390	0.579
24	0.521	0.812	0.57	0.30	....	....	....	0.64	0.53	0.590	0.364	0.561
25	0.500	0.801	0.52	....	....	1.0	....	0.60	0.50	0.572	0.337	0.542
30	0.410	0.735	0.36	....	....	0.95	1.0	0.40	0.33	0.487	0.205	0.451
40	0.273	0.610	0.20	....	....	0.79	0.86	....	....	0.316	.....	0.268
50	0.200	0.500	0.13	....	1.0	0.64	0.52	....	....	0.145	.....	0.095

Two phenomena which concern directly the failure of spirally reinforced columns have not been considered by the author: The method of failure of concrete, and the contraction in volume on drying out. Concrete specimens, whether they fail in compression, tension, or shear, may fail either by the separation of the individual units making up the concrete (grains of sand or stone), due to the overpowering of the cohesive forces in the cement, or by rupture of the individual grains. From observation of several thousand specimens, tested under the speaker's supervision in the concrete laboratories

* *Proceedings*, Am. Soc. C. E., February, 1923, p. 194.

of the University of Cincinnati, he is convinced that failures of the latter type may be disregarded, especially in considering the strength of the usual concrete. Failure of concrete columns is either by flexure or by shear; in either case, the column fails because of the low tensile strength of the concrete. For the tensile strain ruptures the extreme fiber in flexure, and simple shear is a tension along one of two rectangular axes, with a simultaneous compression of equal magnitude along the other. Failure, therefore, occurs whenever the tensile stress overcomes the adhesion produced by the cement. After such failure occurs, the body is no longer a homogeneous solid, even though the separation of the parts may be by an infinitesimal distance. Such failure is reported by several of the experimenters:

"Generally the first sign of failure in the columns appeared in the form of longitudinal cracks, usually occurring from 0 to 2 ft. from one end, although sometimes extending the entire length" (Aberthaw).

"No cracks or other signs of failure were visible until the maximum load was reached, except that in Column 7*a* a small longitudinal crack was seen in the shaft just below the head at a load of 2 190 lb. per sq. in. At the maximum load longitudinal cracks developed all around the shaft of the column below the top of the head and extended down over the upper quarter of the length of the shaft. The load at once fell off 20% or more."*

"In some cases no cracks were visible below the maximum load. In general, the load fell off considerably a half minute or so after the maximum load was applied."†

It has been settled definitely that concrete as it sets in air contracts. A. T. Goldbeck, Assoc. M. Am. Soc. C. E., establishes the contraction at a maximum of 0.0005 at 3 months.‡ The importance of this contraction in the spirally reinforced column is the setting up of initial stresses in the concrete and a compression in the spiral. Considering the effect on the concrete, and assuming that the deformation is equal in all directions, the dilatation, σ, equals $s_x + s_y + s_z = 3\,s$, for an isotropic body, neglecting infinitesimals of higher order than the first. Therefore, s is $\frac{0.0005}{3}$, or 0.00017. This value is probably excessive, for it produces a rather large stress. The normal unit stress, p, causing this dilatation is $(3\lambda + 2\mu)\frac{\sigma}{3}$, where μ is the modulus of shear or of rigidity; σ is the dilatation; and $\lambda = \frac{2\mu}{m-2}$, $\frac{1}{m}$ being the Poisson ratio. (Poisson and others assumed that $\lambda = \mu$, and obtained $\frac{1}{m} = 0.25$; but this is only a special value.) The relation between these constants and E, Young's modulus of elasticity, is given by the equation: $\mu = \frac{m\,E}{2\,(1 + m)}$.

The values of the normal stress, p, for values of E and m, are as given in Table 16.

* Am. Concrete Inst., 1915, on plain concrete columns.

† Am. Concrete Inst., 1915, on columns with rods only.

‡ Am. Soc. for Testing Materials, Vol. XI, "Contraction of Concrete".

TABLE 16.

E.	*m.*	*μ.*	*λ.*	*σ.*	*p*, in pounds per square inch.
2 000 000	0.25	800 000	800 000	0.0005	667
2 000 000	0.20	833 000	555 000	0.0005	556
1 600 000	0.20	667 000	444 000	0.0005	445

The effect of such an internal stress cannot be disregarded. The important factor is that the concrete shrinks away from the spiral; in addition, the fire-proofing outside the spiral puts an initial compression in the steel, the amount of which can be found by the method of computing stresses in pipes under radial pressure, assuming the spiral to act as a steel cylindrical unit. Such action would require a spirally reinforced column to behave in a different way under loadings which cause a strain less than the ultimate than under higher loads. This is easily seen from the tests of such columns. The ultimate strain of plain concrete is less than 0.0015; Walker finds 0.00142 as a maximum strain, with 5000 as the corresponding stress.* To save space, the tests by Withey† are averaged to give Table 17.

TABLE 17.

Number of columns.	Percentage of spiral.	Percentage of rods.	Type of concrete.	Stress at Strain of:				
				0.0005	0.0010	0.0015	0.0020	0.0025
8	0.5	0 to 6.1	1 : 2 : 3½	1 480	2 510	3 200	3 410	3 540
8	1.0	0 to 6.1	1 : 2 : 3½	1 410	2 430	3 000	3 200	3 300
2	1.0	8.0	1 : 2 : 4	2 600	4 700	6 050	6 380	6 500
6	2.0	8.0, 10.0	1 : 2 : 4	2 400	4 400	5 600	5 670	5 750
4	1.0	0 , 6.0	1 : 1 : 2	2 300	4 250	5 500	6 200	......
2	1.0	0 , 5.7	1 : 1¾ : 3¼	1 650	4 100	5 600	6 100	6 450
2	1.0	5.8	1 : 3 : 6	1 750	2 850	3 350	3 500	3 650

It is very noticeable that practically 90% of the ultimate strength of the column is developed by a strain of 0.0015. The speaker believes that the action and failure of spirally reinforced columns is somewhat as follows: As the column sets, the concrete core shrinks away from the spiral, and the outside concrete shell compresses the spiral. As the load is placed on the column, the compressive strain causes a lateral expansion (Poisson's ratio) until a strain of about 0.0010 has occurred. Beyond that strain, the concrete core, as it expands laterally, presses against the spiral balancing and, as it increases, overbalances the initial compression in the spiral. At an axial strain of 0.0015, the concrete has developed its maximum stress and fails; that is, the adhesion is broken. This adhesion is aided somewhat by the tension which now exists in the spiral. There is now a drop in the load as the concrete shifts its burden on

* *Bulletin 5*, Structural Materials Laboratory, 1920.
† Univ. of Wisconsin, *Bulletin 466*.

the spiral. Failure of adhesion does not occur, at first, along more than one shear plane. Additional loading tends to spread the column laterally, which is prevented by the tight steel spiral, and axially, which is opposed by the friction between the separated surfaces. This action is very similar to that of a loaded cylinder of sand encased in a sheet of paper. Axial loads tend to spread the sand particles, but a surprisingly large load is required to cause failure. The speaker at one time had occasion to use paper cylinder forms for concrete test specimens, and noted an increase in strength in the specimens tested with the forms over that of the cylinders made in steel forms.

To quote from the 1915 Report of the American Concrete Institute, the final step is:

"As the maximum load was approached, the spiral stress was no longer uniform around the circumference and it varied at different parts of the length of the column. Finally at some point the maximum strength of the spiral reinforcement was reached or the action of the column became very far from uniform, and the maximum column load which could be carried was attained."

From the foregoing considerations, the theory of elasticity* cannot be used unless there are taken into account: The end conditions, the intial stress in the concrete before loading, the expansion of the concrete bringing the spiral into play, and the incipient separation of the concrete at a strain of 0.0015. Beyond that point, the mass is so far from homogeneous that the theory of elasticity cannot apply.

The author has refused to admit the accuracy of the measurements which gave an evident Poisson's ratio greater than 0.5. In an isotropic body that would be impossible. The author claims that the method of measurement, the axial strain being measured over the entire length and the lateral strain localized, is the reason for such values. In the American Concrete Institute columns, lateral and axial strain at twenty points on the concrete and on the spiral were determined. The following gives the average of all the readings in the test of Column 3*b*, 20.75 in. in diameter, 1 : 1½ : 3 concrete, 1.89% vertical and 0.94% spiral steel, and no fire-proofing cover:

Axial unit load..............	3 500	4 350	5 000	5 450	5 800
Longitudinal strain.........	0.001	0.0014	0.0025	0.0035	0.0050
Lateral strain..............	0.0002	0.0003	0.00075	0.0015	0.0027
Poisson's ratio.............	0.20	0.24	0.30	0.43	0.54

Poisson's ratio increases very rapidly after the axial strain has passed 0.0010; in other words, we are no longer dealing with a solid. Too much reliance cannot be placed on the theoretical investigation, even if the stress is below the ultimate, because concrete is much more heterogeneous than is usually assumed. This can be seen best from the difficulty of determining the constants of the material.

It has been proved that E is a variable depending on the conditions under which the concrete was made. Withey reports that at one-fourth of the ultimate stress, n, or $\left(\frac{E_c}{E_s}\right)$, the E_s being fairly constant varied from 9.2

* *Proceedings*, Am. Soc. C. E., February, 1923, pp. 195–197.

to 15 in the columns tested. At the yield point, the variation was from 14 to 30. The $\frac{1}{m}$ was 0.090 to 0.085 at one-fourth of the ultimate stress, and varied with the load. The shear modulus is a function of E and m; the bulk modulus is a function of the shear modulus and m. The dilatation is the ratio of the normal stress to the bulk modulus.

Before the proper theory for columns can be developed, a sufficient number of these "constants" of concrete must be determined experimentally in order to be able to evaluate every one of them.

The author is to be complimented on his exhaustive treatment of the subject, and the extremely interesting form and manner in which it is presented. It opens up several new views and methods of attack which will lead to valuable results in placing concrete design on a more scientific basis.

A. W. Buel,* M. Am. Soc. C. E.—A paragraph on page 173† reads, in part: "As unreinforced concrete columns are never used in practice, tests of specimens of this type are considered of small value. Test results of only eleven such columns can be found." The author's explanation of this statement is that: "They are forbidden by the code." As his opening statement reads: "The purpose of this paper is primarily to determine, by scientific methods, values of safe working stresses" and refers to the Joint Committee's specifications, but not to any "code," his explanation seems to be inconsistent, and, in the speaker's opinion, the limitation is unscientific, restricts the application of the results, and reduces the value of the paper.

A prominent engineer has said that we have to have codes, on account of the ignorant and incompetent designers extant. There is something in that, but do not other professions also need a code as much or more? In the medical profession a really efficient code would have fully as great opportunities of saving lives as in engineering.

Codes are subject to revision and radical changes, as are also specifications and "reports of Committees," both of which are the products of compromise and majority vote. If the speaker may paraphrase the description by the late James J. Hill, F. Am. Soc. C. E., of E. H. Harriman's standard box car:

"The Committee's specifications were written by engineers in conference. Each man had his pet mixture, reinforcing system, column formula, etc. So your specification is a composite of engineers' hobbies, not a truly standardized or scientific specification to meet conditions. A scientifically drawn specification may be written which will be far broader in its application and give greater economy without sacrificing safety; but you can bet such a specification will not be written in conference."

Therefore, provisions of codes and specifications should not be permitted to have a controlling effect on research work.

The author could only find eleven tests of plain concrete columns. This is only one among many instances that illustrate how valuable work of only fifteen or twenty years ago is either entirely overlooked or else ignored by present-day workers. Possibly they think the old experimental work was not

* Cons. Engr., New York City.

† *Proceedings*, Am. Soc. C. E., March, 1923.

sufficiently scientific and accurate to be entitled to serious consideration. Perhaps it is an effect of the World War; but, whatever the cause, research workers would do well to start each new line of work with an exhaustive search for all past experimental results bearing on the subject in hand. In this case, it happens that the U. S. Government publications, "Tests of Metals and Other Materials" made on the testing machine at Watertown Arsenal, for the six years from 1904 to 1909, inclusive, contain reports of tests of 206 concrete columns, plain and reinforced, of which 64 were of plain concrete or mortar. These series are very interesting and instructive. The earlier ones were included in a work by the speaker, published in 1906, which has had wide distribution. They show conclusively that cement is cheaper than steel for reinforcing concrete columns; that a very rich concrete column, without reinforcement, will do the work for less cost than a column of lean concrete reinforced with steel. They also show the low efficiency of hooping for rich concrete. There are also reported in the same volumes nineteen tests of columns with various mixtures, made for the purpose of determining the load sustained by the concrete, and the load sustained by the steel, in columns with longitudinal rods. The author seems to give the impression that these determinations would be difficult to manage. It would seem to be a comparatively simple matter to make such determinations with extensometers.

In many places, the use of plain concrete columns is perfectly good, and gives the greatest economy, but certainly steel reinforcement should always be used where there is eccentricity of loading, transverse loading or bending, or where the beam and girder connections and details make the latter continuous with the column, consequently throwing bending into the column, and when wind is carried by bending in the columns. Thus, in city work, where buildings of several or many stories constitute the bulk of the structures, longitudinal reinforcement will be generally necessary, and it may be quite proper to require it by the code, but in many places far afield the requirements are very different.

The restriction of plain concrete columns to a length of six diameters does not seem to be justified. Plain concrete is, or can easily be made, as good as stone, and lengths of nine to ten diameters occur in several of the classical orders. The speaker thinks ten to twelve diameters should be permitted in certain types of structures and under proper conditions.

Referring to Fig. 2,* the author gives a "representative line" for the relation of strength of columns with slenderness ratios $\left(\frac{L}{D}\right)$ from 0 to 18, to "cylinder strength." The line does not appear to fit the points plotted from the test records, and is not convincing. Although it would be preferable to have a great many more points to locate such a line with any assurance, as far as they go, they certainly indicate a line coincident with the 100% abscissa from $\frac{L}{D} = 0$ to $\frac{L}{D} = 10$, or greater. Moreover, when a larger number of experimental determinations are available, it is not improbable

* *Proceedings*, Am. Soc. C. E., February, 1923, p. 174.

that additional light may be shed on the "discrepancy", mentioned by the author, that his "representative line" does not cross the 100% line at $\frac{L}{D} = 2$, if, in fact, the discrepancy does not entirely disappear. If 100% were used for values of $\frac{L}{D}$ up to 10 or 12, it would be in accord with past practice with other materials.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DECATUR AXTELL, M. Am. Soc. C. E.*

DIED NOVEMBER 27, 1922.

Decatur Axtell, the son of Almon and Sophronia (Boynton) Axtell, was born at Elyria, Ohio, on February 8, 1848. He was descended from a near relative of Daniel Axtell who was styled one of the twelve "Regicides", who condemned Charles I to death, ten of whom, including Daniel Axtell, were hung at Tyburn. Daniel Axtell was a Colonel in the Army of Oliver Cromwell and an intimate friend of Cromwell and his family. The Axtells, from whom Decatur Axtell was descended, were loyal to the Crown, and pronounced Daniel Axtell a fanatic; but the ignominy of Tyburn, and the disfavor of the name thereafter, drove one of the Royalist Axtells to join the "Pilgrim Fathers", which accounts for the Massachusetts ancestry of Decatur Axtell. His mother, through the Delanos on her mother's side, traced her lineage to Philip de la Noix of France.

Mr. Axtell received his early education in Ohio, and during 1866 and 1867, he was a student at Illinois College.

He began his railroad career on March 16, 1864, when he was engaged as Rodman on the Pacific Railroad of Missouri. In 1865, he was promoted to be Assistant Engineer of Construction on a division of the line south of Leavenworth, Kans.

After a year at Illinois College, Mr. Axtell became Engineer of Construction, Tunnel Division, of the St. Louis and Iron Mountain Railway, and held this position from October, 1867, to August, 1869. From the latter date until January, 1872, he served as Assistant Engineer of the same road and also of the Cairo and Fulton Railroad.

In January, 1872, he became Assistant Engineer of the St. Louis, Iron Mountain and Southern Railway and Chief Engineer of the Cairo, Arkansas and Texas Railroad, and held these positions until January, 1875. For a period of five years, until July, 1880, he served as Assistant Engineer and Division Superintendent of the St. Louis, Iron Mountain and Southern Railway.

In July, 1880, Mr. Axtell was appointed General Manager of the Richmond and Allegheny Railroad, which was then being constructed, and remained in that position until April, 1882, when he was appointed Vice-President and General Manager. He served in this capacity until June, 1883, when the Company was declared insolvent and he was appointed Receiver and Manager. At the end of his Receivership in 1889, he was elected Second Vice-President of the Chesapeake and Ohio Railway, which Company had acquired the Richmond and Allegheny Railroad.

* Memoir prepared by James Poyntz Nelson, Valuation Engr., Cheapeake & Ohio Ry. Co., Richmond, Va.

Mr. Axtell then began the task of rehabilitating the property of the Chesapeake and Ohio Railway Company. In February, 1900, he was appointed Vice-President, holding this position until February, 1918, when he retired. He was so closely connected with the work of the Chesapeake and Ohio Railway Company that almost every part of the System bespeaks of his prescience, ability, and professional masterliness.

He was also prominently connected with other railroads. From 1899 to 1903 he was President of the Toledo and Ohio Central Railway and Vice-President of the Kanawha and Michigan Railway. From 1903 to April, 1909, he was Chairman of the Board of Directors of the Toledo and Ohio Central Railway, and from 1903 to 1910 served in the same capacity with the Kanawha and Michigan Railway. In April, 1910, he became Vice-President of the Hocking Valley Railway, his services as such terminating when he retired as Vice-President of the Chesapeake and Ohio Railway.

Mr. Axtell died suddenly on November 27, 1922, at his home in Richmond, Va. He was married to Miss Ellen May Cantrell, a daughter of Dr. William A. Cantrell, of Little Rock, Ark., who survives him.

Mr. Axtell was known as a reticent man. He was called "the silent member of the Board of Directors." In business affairs, he moved with caution, realizing his responsibility, for on his judgment frequently rested the success of vast undertakings and the expenditure of large sums of money. Therefore, his advice had great weight with those who trusted him with the management of their affairs. He reasoned with meticulous exactness and was always ready to defend his conclusions, as well as to listen politely to the opinions of his opponents.

He was an engineer who thought in the finest categories of that profession and his methods bespoke his engineering training. He was also an able executive with capacity for directing large enterprises; but more than this he was a gentleman who never failed in perfect courtesy, even under the most trying circumstances, a man who with utter directness ever gave the best that was in him and who by a gentleness and courtesy all too rare, and with fine intelligence, made the obeyance of his orders a pleasure to those under him.

He was President of the Virginia Hot Springs Company from 1891 to 1911, and of the White Sulphur Springs Company from 1911 to 1918. He was also a member of the Virginia Historical Society, the South Carolina Historical Society, the Society of Mayflower Descendants, and the Society of Colonial Wars.

Mr. Axtell was elected a Member of the American Society of Civil Engineers on March 3, 1886.

CHANNING MOORE BOLTON, M. Am. Soc. C. E.*

Died December 11, 1922.

Channing Moore Bolton, the fourth son of Dr. James and Anna Maria (Harrison) Bolton, was born at Richmond, Va., on January 24, 1843. His

* Memoir compiled from information on file at the Headquarters of the Society.

father, Dr. Bolton, was born in Savannah, Ga., in 1812, and was a graduate of Columbia College and the College of Physicians and Surgeons of New York City. He afterward studied theology and was ordained as a clergyman of the Protestant Episcopal Church, but returned to the practice of medicine in Richmond after a brief period spent in the ministry. The Bolton family is of English origin. Its pedigree has been traced to a period following the Norman Conquest when the family possessed large estates in Yorkshire and Lancashire.

Channing Moore Bolton received his early education at several private primary schools and at the preparatory school conducted by Mr. William D. Stuart in Richmond. In 1860, he entered the University of Virginia where he studied Latin, French, and mathematics. In 1861, he joined one of two student companies from the University that entered the Confederate Army, and was engaged in constructing fortifications around Richmond and in charge of building three forts near Brooke Turnpike.

In 1862, he was employed on the construction of a railroad to fill a gap between Danville, Va., and Greensboro, N. C., and continued in the railway engineering service holding the positions of Rodman, Transitman, and Resident Engineer in the Confederate Army. In the spring of 1863, he was commissioned a Lieutenant of Engineers in the First Regiment of Engineer Troops and ordered to report to Major-General Pender, of A. P. Hill's Army Corps, to act as Engineer Officer on his Staff. He joined General Pender at Winchester, Va., and accompanied him on the campaign into Pennsylvania where he took part in the Battle of Gettysburg.

Lieutenant Bolton assisted in the construction of the pontoon bridge across the Potomac River over which the Confederate Army crossed on its return to Virginia. He was in charge of the bridge when the crossing took place and also caused it to be destroyed when the Confederate troops were safely over, just before the arrival of the Federal forces. He took part in most of the battles fought by the Army of Northern Virginia.

Mr. Bolton continued in the Army until the end of the war in 1865, and at this period suffered many hardships and privations. After a few months spent in Southwestern Virginia, he returned to Richmond. In 1866, he surveyed, located, and constructed the Clover Hill Railroad in Virginia. In the latter part of 1866, and in 1867, he was engaged in constructing the tunnel at Richmond for the Richmond, Fredericksburg, and Potomac Railroad, and from 1867 to 1869 he was Resident Engineer of the Louisville, Cincinnati, and Lexington Railroad.

From 1869 to 1874, he was Engineer of the Chesapeake and Ohio Railway in charge of construction of a part of the road between Covington, Va., and White Sulphur Springs, W. Va. This was one of the heaviest pieces of railroad work constructed to that time. Mr. Bolton then organized and was in charge of a party of engineers, which located the road down New River in West Virginia, a very difficult enterprise. After locating the line from Richmond to Newport News, Va., he located and constructed a double-track tunnel ¾ mile long, under Church Hill, Richmond, which arduous undertaking was successfully accomplished.

From 1876 to 1879, Mr. Bolton was United States Assistant Engineer in charge of the canal around the Cascades of the Columbia River in Oregon, and designed all the plans for this work. In 1879 and 1880, he was Division Engineer of the Richmond and Allegheny Railroad and he also had charge of the location and construction of that road from Richmond to Lynchburg, Va., which involved the changing of the old James River and Kanawha Canal into a railroad.

From 1880 to 1881, he served as Engineer and Superintendent of the Greenville (Miss.), Columbus and Birmingham Railroad, and from 1882 to 1895, he was Chief Engineer of the Richmond and Danville Railroad and of the Southern Railway.

During 1879 and 1880, Mr. Bolton was President and Manager of the Richmond City Street Railway which included all the street railways in Richmond at that time and of which he finally became sole owner. He operated these lines with great efficiency and eventually sold them at five times the original cost.

Mr. Bolton resigned as Chief Engineer of the Southern Railway in 1895, and moved from Washington, D. C., to his farm in Albermarle County, Virginia, engaging in private practice as a Consulting Engineer. In May, 1907, under contract, he built two tunnels near Garrison, Mont., one a double-track tunnel for the Northern Pacific Railroad Company, and the other for the Chicago, Milwaukee, and St. Paul Railroad Company.

Among his numerous activities may be mentioned that of President of the Charlottesville Street Railway Company and Charter Member and Director of the Charlottesville Ice Company. He was also President of the Meadow Creek Country Club, Trustee of the Miller School Board, member of the Miller Board of the University of Virginia and of the Executive Committee of the University of Virginia Alumni Association, and a Trustee of St. Paul's Chapel, University of Virginia. He drew the plans and supervised the construction of the beautiful Church of Our Saviour at Rio, Va., of which he was also a Trustee.

In 1911, Mr. Bolton became Director of the People's Bank of Charlottesville, Va., and in 1913, he was elected President of the Miller Board of the University of Virginia. He was also President of the Board of Trustees of St. Anne's School at Charlottesville and Chairman of the Highways Committee of the Charlottesville Chamber of Commerce. During the World War, he served as Chairman of the Local Board. In every capacity, he worked for the good of this locality.

Mr. Bolton contributed papers and reports to the Reports of the Chief of Engineers of the United States Army for the years 1877, 1878, and 1879.

He was married on February 17, 1874, to Miss Lizzie Calhoun Campbell, of Richmond, who died on October 6, 1889. His second wife, whom he married on June 6, 1894, was Miss Alma Ann Baldwin, of Montgomery, Ala. He is survived by his widow, his daughters, Mrs. J. Thompson Brown, Jr., of Rock Hill, S. C., Mrs. W. Allan Perkins, of University, Va., and Miss Cecile Baldwin Bolton, his son, Channing Moore Bolton, and his brother, Dr. Meade Bolton.

His constructive ideals and deep interest in the welfare of the community in which he lived will stand as a memorial of his worth, and the accomplishments of his vast energy are an index of his desire to serve his fellow men. Nothing deterred him from unswerving fidelity and faithful service to the cause of the Protestant Episcopal Church, and devotion to his family and loyalty to his friends were characteristics of his true personality.

Mr. Bolton was elected a Member of the American Society of Civil Engineers on January 4, 1888.

SHIRLEY CARTER, M. Am. Soc. C. E.*

DIED JANUARY 5, 1923.

Shirley Carter was born in Richmond, Va., on January 4, 1869. He was a lineal descendant of the original Carters, who settled in Virginia early in the Seventeenth Century.

He first entered the Government service on August 16, 1888, serving in various grades in the Baltimore, Md., District of the Engineer Department, United States Army, until October 19, 1892. From October 25, 1894, to November 30, 1895, he was employed in various capacities in the Wilmington, Del., District. From September 23 to December 7, 1896, he appears on the rolls of the Montgomery, Ala., office. He then worked in a private capacity until October, 1902, on which date he re-entered the Federal service in Norfolk, Va., and served continuously in this District up to the time of his death. On January 1, 1907, he was promoted to the final and highest grade in the service, that of Assistant Engineer.

At the time of his death, Mr. Carter had immediate charge over all the river and harbor improvements in Norfolk Harbor and tributaries, the Inland Waterway from Norfolk to Beaufort, N. C., the improvement of Thimble Shoals Channel, and of the Nansemond and Pagan Rivers. These projects involved the expenditure of many millions of dollars, and their success has been due in no small measure to Mr. Carter's energy, foresight, good judgment, integrity, and unswerving loyalty to duty. He also supervised the operation and repairs of all floating plant in the Norfolk District. In this work he was especially capable and well qualified, on account of his long experience in the operation and repair of floating plant of all kind.

To quote from one who knew him well, as a friend and as a co-worker:

"Mr. Carter was considered by all the employees who had worked with and under him as a capable, hard working, conscientious engineer; was a man who attended strictly to his own business, and was always ready and willing to lend a helping hand to his fellow man."

His early death removes from the public service a loyal, rugged and true-hearted employee, and leaves with those who were privileged to enjoy his friendship a keen sense of the loss of a friend who never failed in time of need.

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* Memoir prepared by J. P. Jervey, City Mgr., Portsmouth, Va.

Mr. Carter was elected a Junior of the American Society of Civil Engineers on May 31, 1892; an Associate Member on October 5, 1898; and a Member on December 4, 1906.

RICHARD HENWOOD GILLESPIE, M. Am. Soc. C. E.*

DIED JULY 15, 1921.

Richard Henwood Gillespie was born in New York City on March 17, 1867, his parents having been William B. Gillespie and Elizabeth Henwood Gillespie. After completing his common school course, he entered Union College, Schenectady, N. Y., and was graduated with the degree of Civil Engineer.

From 1889 to 1891 Mr. Gillespie was engaged on railway reconnaissance, location, and construction in Tennessee, Virginia, and North Carolina, as Instrumentman, Topographer, and Resident Engineer. From 1891 to 1893 he was Engineer for the contractors on the Little Falls and Dolgeville Railroad, Little Falls, N. Y. In 1893 and 1894 he was Division Engineer on the location and construction of the Eastern Division of the Castagena-Magdalena Railroad, Colombia, South America. In 1894 and 1895 he was again Engineer for contractors, this time on Sections 10 and D of the Chicago Drainage Canal.

From 1896 to 1907 Mr. Gillespie was continuously in the service of the City of New York, as Transitman and Assistant Engineer in charge of construction in the Borough of the Bronx. From 1907 to 1910 he was Engineer for a contracting firm on the construction of a large sewer outlet in the Borough of the Bronx, and a section of the Catskill Aqueduct. During this engagement he designed a concrete form for sewers and aqueducts which was patented and adopted by a well-known firm of concrete form manufacturers.

From 1910 to 1918 Mr. Gillespie was Chief Engineer of Sewers and Highways, Borough of the Bronx, New York City, during which time from $2 000 000 to $3 000 000 was expended on highway and sewer work each year. In 1918 he took charge of the construction of sanitation work at Nitro, W. Va., for the smokeless powder plant there, and on January 1, 1919, he became Chief Engineer and General Manager of the Traylor-Dewey Construction Company, doing cement gun work principally. In the spring of 1921, he contracted a serious illness which resulted in his death on July 15, 1921.

Mr. Gillespie was married in 1896 and left a widow, Mrs. Hortense J. Gillespie, three sons, Harold, Richard, and Robert, and a daughter, Elizabeth.

His friends and associates sustained a great loss in his death, having appreciated his sterling character, engineering ability, and painstaking application to his professional work.

Mr. Gillespie was elected an Associate Member of the American Society of Civil Engineers on October 3, 1900, and a Member on June 5, 1906.

* Memoir prepared by R. A. MacGregor, M. Am. Soc. C. E.

HERBERT THOMAS GRANTHAM, M. Am. Soc. C. E.*

Died November 4, 1922.

Herbert Thomas Grantham, son of Thomas and Anna (Bootes) Grantham, of England, was born at Athens, Pa., on September 15, 1868, and received his early education in the elementary and High Schools of his native town.

After his graduation from the High School, he immediately entered the employ of the Union Bridge Company, of Athens, Pa., one of the first companies in the United States to construct all-steel bridges. He remained in the service of that company several years, and then accepted more responsible duties with the Edge Moor Bridge Works, near Wilmington, Del.

Later, he was stationed in Philadelphia, Pa., where he was associated with some of the most prominent bridge and structural steel companies of the East, and in 1898 became Chief Engineer of the Belmont Iron Works. His energy, courage, and devotion to duty were inspiring to all with whom he came in contact, and his wise counsel added much to the prestige and growth of the company. In 1918 he was made its Vice-President.

He was known to his business associates as a man of sterling character, loyal to his friends, of pleasing personality, and excellent business judgment.

In the spring of 1893 he was married to Miss Alice Burton South, who, with a son, Leslie Burton Grantham, and a grandson, survives him.

Mr. Grantham was elected an Associate Member of the American Society of Civil Engineers on February 5, 1896, and a Member on May 7, 1902, and at his death held membership in the Union League of Philadelphia, Philadelphia Country Club, and the Aronimink Golf Club.

MACE MOULTON, M. Am. Soc. C. E.†

Died April 27, 1909.

Mace Moulton was born in Manchester, N. H., on February 15, 1855, of a distinguished family, his father being a man of great scientific attainments, and his grandfather, whose name he bore, at one time a Congressman from New Hampshire. In 1872, Mr. Moulton entered the preparatory department then maintained by the Thayer School of Civil Engineering, on the discontinuance of which his preparatory studies were made in the Chandler Scientific Department. His studies were interrupted from time to time when he engaged in professional work.

After his graduation, and for six months, until March, 1870, he was Assistant Engineer in the Department of Maintenance of Way of the Eastern Railroad, at Salem, Mass. Then, for most of the time during the next four years

* Memoir prepared by George S. Webster, Past-President, Am. Soc. C. E.

† Memoir compiled by Robert Fletcher, M. Am. Soc. C. E., the first part rfom an obituary in the Dartmouth *Bi-Monthly* for June, 1909; the second part from correspondence and data in the office of the Thayer School of Civil Engineering.

he was associated with the late C. Shaler Smith, M. Am. Soc. C. E., of St. Louis, Mo., the noted bridge engineer, who designed and built the first cantilever bridge in the United States. During this period Mr. Moulton designed numerous important railroad bridges.

From June, 1883, to December, 1884, he served as Principal Assistant Engineer to the Edge Moor Iron Works, Wilmington, Del. In 1885 he was Chief Assistant in the construction of the Kentucky and Indiana Bridge over the Ohio River at Louisville. In 1886-87 he was for two years Engineer of Bridges of the Colorado and Midland Railway, designing and superintending the construction of all the bridges and buildings of the road between Colorado Springs and Leadville, Colo. For a year and a half, in 1888-89, he was in private practice in Boston. In 1889-90, he was for two years Consulting Engineer of the Berlin Iron Bridge Company, of East Berlin, Conn.

From 1891 to 1896 Mr. Moulton was Chief Engineer of the R. F. Hawkins Iron Works, at Springfield, Mass., building many important railway and highway bridges and many iron and steel frames for buildings. From 1896 to 1903 he was engaged in private practice in Springfield, designing and constructing many steel structures throughout New England, and acting in a consulting capacity for various cities and railroads.

Since 1903 Mr. Moulton was a Consulting Engineer in New York City. Among his tasks was the reconstruction of the bridge across the Hudson River, at Poughkeepsie, an exceedingly difficult and complicated piece of engineering, which was accomplished successfully, the expenditure, nearly $1 500 000, being kept within the original estimate. In 1907 he was made President and Chief Engineer of the Milbrook Company and its Allied Companies, capitalized at $20 000 000, and aiming to provide rapid transit for the northern part of New York City and the neighboring parts of Westchester County. This position he held at the time of his death.

Mr. Moulton's first wife, Emma Blaisdell, of Hanover, N. H., died during their residence in Springfield, and, later, he married her sister who survived him. There were three children, Mace, Jr., who was associated with his father in business, Thornton, a student at the University of Pennsylvania, and Mrs. Walter O. Greene, of Wakefield, N. J.

It is appropriate to add a few facts concerning the earlier years of Mr. Moulton's career: When a student he often surprised the writer by a certain degree of precocity (he looked younger than he was) when he betrayed unexpected aptitude in his grasp of engineering principles and details of practice. He was a very neat draftsman and penman, accurate in computation, and had a fine artistic sense. When the office procured one of Edison's "electric pens" (the precursor of the mimeograph) for manifolding purposes, he designed and printed an ornate letter head which showed his artistic skill, with an entirely new implement.

Mr. Moulton's graduating thesis, on the Haverhill (Mass.) Drawbridge, then just finished, was a model in completeness and execution. With few suggestions from his instructor, he made the measurements by himself on the spot, all the elaborate calculations, full small-scale drawings showing many of

the details, and wrote a clear description of the work. This unusually complete description and careful analysis was presented in about 120 manuscript pages and more than 25 diagrams and colored drawings. Undoubtedly, this was the means of his receiving, about a year later, an offer from Mr. C. Shaler Smith, to take him as an Assistant. During his course of training under this distinguished bridge designer, of whom he became an apt and devoted pupil, Mr. Moulton occasionally communicated with the writer telling of his work, and sent interesting notes, sketches, and descriptions exhibiting Mr. Smith's methods and several novel and original features of designs from the master mind. Among these was Mr. Smith's novel device for hanging the railroad in the Royal Gorge of the Arkansas River from iron rafters spanning the narrow stream; also Mr. Smith's predilection for the use of the continuous girder in certain cases; and his ingenious use of a link in the end panel of double-intersection trusses to distribute automatically the stress in the two systems so as to diminish the ambiguity. He told the writer afterward that Mr. Smith "believed in young men." Certainly that was so in this case, for Mr. Moulton was entrusted with the entire design of the Sabula Drawbridge, under the governing conditions imposed by Mr. Smith. The notes and blueprints relating to this work constituted an admirable example of the very best practice in bridge design at that date.

It was during the student days of Mr. Moulton that the method of graphic statics was brought to the notice of the Profession by the late Professor A. J. DuBois, M. Am. Soc. C. E., of Yale, through *Van Nostrand's Engineering Magazine*. Mr. Moulton at once became an enthusiastic disciple in mastering the principles.

Just before his death, Mr. Moulton was elected President of the Thayer Society of Engineers, of Dartmouth College. In later years, he gave the impression of being quick to assert the rights, dignities, and standing of his Profession, an example which, if more generally followed, would enhance the general and public recognition which should be accorded to it.

Mr. Moulton was elected a Member of the American Society of Civil Engineers on June 4, 1884.

ROBERT MORRIS NEWMAN, M. Am. Soc. C. E.*

DIED NOVEMBER 13, 1922.

Robert Morris Newman, the son of Edward and Eliza Newman, was born in Cheltenham, England, on August 1, 1839. He was educated at the University of Montreal, Canada, and on January 8, 1859, was articled for three years to Joseph William Burke, a practicing Civil Engineer and Provincial Land Surveyor, of Canada. He practiced afterward as Civil Engineer and Surveyor until July, 1865.

From July to November, 1865, Mr. Newman was Assistant City Engineer of the City of Fort Wayne, Ind. From December, 1865, until October,

* Memoir prepared by William G. Fargo, M. Am. Soc. C. E.

1868, he served as Assistant Engineer on the Pittsburgh, Fort Wayne and Chicago Railroad. From October, 1868, until April, 1870, he was Resident Engineer on the Massillon and Cleveland Railroad, in charge during the location and construction of 13 miles of road. From April, 1870, until February, 1871, he was Resident Engineer of the Michigan Air Line Railroad (now the Air Line Division of the Michigan Central Railroad) during its construction between Jackson and Niles, Mich., a distance of 103 miles.

From February, 1871, until August, 1873, Mr. Newman was a Resident Engineer and then Principal Assistant Engineer for the Michigan Central Railroad, being stationed at Jackson, Lansing, and at Cheboygan, Mich. During this period the present passenger station at Jackson was built under his supervision, also locomotive shops and houses, water stations, second tracks, etc. The late Henry A. Gardner, M. Am. Soc. C. E., was then the Chief Engineer of the Michigan Central Railroad.

From August, 1873, to some time in 1875, Mr. Newman was Assistant Engineer on the Erie Railway. He then returned to Jackson, where he remained until about 1880 and where he was married to Miss Kate Smith, daughter of Hiram H. Smith. During this period of his residence in Jackson, he was engaged more or less as an Assistant or Resident Engineer with the Michigan Central Railroad.

During 1880 and 1881, Mr. Newman was Assistant Chief Engineer with the Northern Pacific Railroad, being stationed at Jamestown, S. Dak., and Glendive, Mont. In 1881, he went to Minneapolis, Minn., and became interested in the manufacture of newsprint paper, the firm name being Warner, Newman and Elfelt. He remained in Minneapolis until 1887, and then returned to Jackson to become City Engineer, taking office on April 16, 1888. This position he retained until 1902.

From 1902 until 1910, Mr. Newman was Manager of the Jackson Vehicle Company, retiring when this concern was reorganized as the Jackson Automobile Company. He was a Director of the Longyear and Mesaba Land and Iron Company of Minnesota, from about 1895, and made occasional trips to this iron region.

Mr. Newman served from 1903 to 1910 as Vestryman of St. Paul's Protestant Episcopal Church, of Jackson, and was a member of the principal local clubs. He was a dignified and courtly gentleman and a capable business man, as well as a thorough engineer-executive possessed of excellent judgment.

Mr. Newman was elected a Member of the American Society of Civil Engineers on May 6, 1874.

HARRY KENT SELTZER, M. Am. Soc. C. E.*

Died December 30, 1921.

Harry Kent Seltzer, the son of William K. Seltzer and Emma (Keller) Seltzer, was born at Ephrata, Pa., on January 22, 1875. Nearly all his ancestors on both sides were Pennsylvania farmer folk.

* Memoir prepared by L. S. Stewart, Affiliate, Am. Soc. C. E., E. E. Howard, and M. B. Case, Members, Am. Soc. C. E., and S. E. Blum, Esq., Detroit, Mich.

Immediately after his graduation from Lehigh University, where he received the degree of Civil Engineer in 1895, he entered the service of J. A. L. Waddell, M. Am. Soc. C. E., and went to Sioux City, Iowa, as Assistant to Lee Treadwell, M. Am. Soc. C. E., Resident Engineer on the construction of a bridge over the Missouri River. During the early part of 1896 he was Inspector of Superstructure on a bridge over the Missouri River at Jefferson City, Mo. In May, 1896, he entered the employ of the Kansas City Southern Railroad, as Instrumentman and Draftsman on preliminary and location work in Southwestern Louisiana. During a portion of this time he was Inspector on the foundations for the Calcasieu River Bridge at Lake Charles, La., on this line.

From May to November, 1898, he was Transitman on Maintenance of Way work, on the Philadelphia Division of The Pennsylvania Railroad, at Philadelphia, Pa. During the college year 1898-99 he was Instructor in Civil Engineering at the University of Texas, after which he spent six months as Resident Engineer for Waddell and Hedrick on the construction of bridges along the line of the Vera Cruz and Pacific Railway, in the State of Vera Cruz, Mexico. This was during the beginning of construction of that road, and conditions were very primitive. While there, Mr. Seltzer almost succumbed to a very severe attack of yellow fever.

After a year spent in Kansas City as Designer and Contracting Engineer for the Midland Bridge Company, he returned to the employment of Waddell and Hedrick, in April, 1901, with whom he was associated until May, 1906. During this period he was stationed first at Alexandria, La., as Resident Engineer on a bridge over the Red River. In May, 1902, he went to New Westminster, B. C., Canada, in charge of the construction of the Fraser River Bridge. This structure was notable for the main pier foundations sunk by open dredging methods in 60 to 80 ft. of water to a depth of 138 ft. below the water surface, and in a current, at times, extremely rapid.

While at Westminster, Mr. Seltzer was also in charge of work for the Great Northern Railway at Vancouver, B. C., Canada, all of which reflected great credit on his resourcefulness and executive ability. During the year following May, 1905, he was in charge of the reconstruction of ten bridges on the International and Great Northern Railroad, in Texas, where the work was done under traffic.

In May, 1906, Mr. Seltzer was employed by the Missouri Valley Bridge and Iron Company, Contractors for the foundations of the Atchafalaya Bridge, at Morgan City, La., for the Southern Pacific Railway Company. Here he was in charge of sinking pneumatic caissons to a depth of 105 ft. in 70 ft. of water, and with a very soft bottom which developed little supporting friction.

At the conclusion of this work, Mr. Seltzer became a Member of the Union Bridge and Construction Company, and from 1907 to 1912, as Constructing Engineer, he contributed in no small way to the successful execution of many large contracts. At Portland, Ore., he directed the construction of the foundations for the Oregon-Washington Railroad and Navigation Com-

pany bridge over the Willamette River, where the main piers were sunk by open dredging methods in 85 ft. of water to a depth of 133 ft. below the water surface. The total cost of the substructure on this contract exceeded $500 000. During 1911 and 1912 he was stationed in Kansas City, but made frequent trips to Portland where he directed the work on the foundations for the Broadway Bridge over the Willamette River. This substructure consisted of four main piers, sunk by the pneumatic process, and other open foundations at a total cost of $750 000. The contract for the foundations of the bridge over the Yellowstone River, at Mondak, Mont., for the Great Northern Railway Company, was also completed during this time.

Mr. Seltzer's capacity to handle men and materials and to draw about him and hold those whose ability, loyalty, and earnestness were beyond question was now recognized by his promotion to the position of Vice-President and Chief Engineer. In the period from 1913 to 1918 the firm constructed the foundations for three of the largest bridges built in this country during that time—the Harahan Bridge, over the Mississippi River at Memphis, Tenn., the crossing of the Ohio River at Metropolis, Ill., and the Burlington Railroad Bridge over the Missouri River at Kansas City. The complete satisfaction frequently expressed by the officials of the railroads for whom these works were executed reflected in no small degree the untiring zeal and resourceful ability which Mr. Seltzer constantly applied to his work. The reputation for rugged honesty and efficient production which he and the organization he built around him enjoyed has often been a subject of comment, and was shown by the frequency with which he was asked to carry out force-account work for various railroads, with little or no competition.

During the war period, 1917-19, he was stationed at Morgan City, La., as Manager of the Emergency Fleet Corporation's shipyard, and built nine ships, every one of which was put into commission. This yard was a source of great satisfaction to the Shipping Board officials at a time when many were not operating so smoothly.

In November, 1920, Mr. Seltzer severed his connection with the Union Bridge and Construction Company, and represented the Foundation Company of New York at Kansas City.

He died suddenly on December 30, 1921, at Kansas City, Mo., only a few days after his return from a business trip in the East.

One of his friends writes:

"I was Mr. Seltzer's assistant in Alexandria in 1901 and 1902, and we lived together there, and since that time we have been more or less in contact throughout all his work. Throughout his entire career he always commanded the confidence and respect of every one with whom he came into contact. He was so universally esteemed that I cannot recall ever hearing any one say anything derogatory about him, and I do not think I have another acquaintance concerning whom I have not heard some criticism."

Mr. Seltzer was married in November, 1906, to Miss May E. Waters, who, with two children, survives him.

He was a member of The University Club of Kansas City, the Delta Tau Delta Fraternity, and the American Association of Engineers.

Mr. Seltzer was elected a Junior of the American Society of Civil Engineers on February 4, 1896; an Associate Member on January 2, 1901; and a Member on April 3, 1906.

EDWARD BALLINGER TAYLOR, M. Am. Soc. C. E.*

Died November 8, 1922.

Edward Ballinger Taylor was born at Riverton, N. J., on February 6, 1850. He was a son of John Gardiner Taylor and Rebecca Haines (Ballinger) Taylor. His parents were members of the Society of Friends, and he was trained in the tenets of that faith. The ideals of integrity, devotion to duty, and kindliness, inculcated by that training were reflected in his character through his whole life.

Mr. Taylor was prepared for college at the Westtown, Pa., Boarding School, and in 1866 entered the Sophomore Class at Haverford College, from which he was graduated in 1869, receiving the degree of Bachelor of Arts. He then entered the Polytechnic College of the State of Pennsylvania where, in 1870, he was given the degree of Bachelor of Civil Engineering. From the same institution, in 1873, he received the degree of Master of Civil Engineering.

On July 25, 1870, Mr. Taylor entered the service of the Pennsylvania Railroad, as a Clerk in the office of the Superintendent of the Middle Division, at Harrisburg, Pa. His efficient service won rapid advancement, and in September, 1871, he was promoted to the position of Supervisor, and six months later, on March 1, 1872, was appointed Assistant Engineer of the Middle Division. He continued in that office until January 1, 1875, when he was transferred to a similar position on the Pittsburgh Division.

On July 24, 1876, just six years after entering the service of the Pennsylvania Railroad, he was made Superintendent of the Lewistown Division, with headquarters at Sunbury, Pa., where he remained until January 1, 1879, when he was transferred to the Superintendency of the West Penn Division. On September 1, 1879, he was transferred to the Lines West of Pittsburgh, and made Superintendent of the Pittsburgh, Cincinnati and St. Louis Railway. He was promoted on April 1, 1888, to the position of General Superintendent of the Northwest System, and on March 1, 1890, to that of General Superintendent of Transportation of the entire system of the Lines West of Pittsburgh.

On December 27, 1901, Mr. Taylor was elected a Director and Fourth Vice-President of the Pennsylvania Company and of the Pittsburgh, Cincinnati, Chicago and St. Louis Railway Company. On January 9, 1907, he became Third Vice-President, and on February 1, 1914, Second Vice-President, in charge of the Treasury and Accounting Departments. He also served as President, Vice-President, or Director of fifty-two subsidiary or affiliated

* Memoir prepared by W. L. R. Haines, M. Am. Soc. C. E.

companies of the Pennsylvania System. On March 1, 1920, having attained the age of seventy years, he was, in accordance with the provisions of the Pension Department of the Pennsylvania System, retired from active service.

In addition to his duties as an officer of the Pennsylvania Railroad, Mr. Taylor was for more than twenty-five years a Director of the First National Bank of Sewickley, Pa., and was for some time a Director of the Second National Bank of Pittsburgh, Pa., and, after its merger, of the First-Second National Bank of Pittsburgh, Pa.

At the time of his death, Mr. Taylor had been for many years a resident of Sewickley, Pa., and had always taken a keen interest in its needs and activities. Since 1891, he had served as a member, and since 1895, as President, of its Board of Water Commissioners, and it was due largely to his efforts and guidance that the Sewickley water supply was brought to its present status, as one of the best and most economical in the State of Pennsylvania. He was one of the original members of the Board of Trustees of the Sewickley Valley Hospital and of the Sewickley Young Men's Christian Association, and took an active part in the work of both institutions. He had served as President of the Corporators of the Sewickley Cemetery and of the Edgeworth Club. He was a member of the Allegheny Country Club, the Pittsburgh Club, and the Bankers Club of New York. He was also a member of the American Academy of Political and Social Science, and of the Pittsburgh Academy of Science and Art.

He was a Charter Member of the Engineers' Society of Western Pennsylvania, and was President of that Society in 1886, and a Director in 1887 and 1888.

Although Mr. Taylor never forsook his allegiance to the Society of Friends, he was for many years a communicant of St. Stephen's Protestant Episcopal Church in Sewickley. He served as a Vestryman and as a member of the Building Committee of this church, and was largely instrumental in securing the replacement of the old frame building with the present handsome stone edifice. In his later years, he attended the Sewickley Presbyterian Church, of which he was twice elected a Trustee.

Mr. Taylor was married in 1872 to Miss Marianna Satterthwaite, of Oxford Valley, Pa., by whom he is survived. He also leaves three daughters, Mrs. Charles A. Woods, Miss Bertha A. Taylor and Miss Rebecca W. Taylor, all of Sewickley, and one son, Edward B. Taylor, Jr., Assoc. M. Am. Soc. C. E., Superintendent of the Marietta Division of the Pennsylvania Railroad, with headquarters at Cambridge, Ohio.

It seems not inappropriate to close this memoir with a quotation from the editorial columns of a local paper—a fitting tribute to the regard in which Mr. Taylor was held by those among whom he lived and worked:

"There are but few men in the Sewickley Valley whose death would be so great a public loss as is that of Edward B. Taylor. Known and liked in a personal way by every one, he was regarded by all with a most unusual respect because of his long and valuable service to Sewickley and to the Valley. The same skill, knowledge, broad vision, and safe judgment which brought him pre-eminence in the management of a great railroad system's affairs, had for many years been placed freely at the service of his home community and

its needs, and have been instrumental in contributing much to its welfare and will contribute much more in the future as improvement plans not yet brought to completion are worked out by other hands. Few men have given so freely, so effectively and so unobtrusively of their time and wisdom in a constructive way to the guiding and betterment of public undertakings in their home communities, and Mr. Taylor's death is a loss indeed to this Valley."

Mr. Taylor was elected a Member of the American Society of Civil Engineers on September 3, 1884.

RALPH BENJAMIN ALLEN, Assoc. M. Am. Soc. C. E.*

Died June 17, 1922.

Ralph Benjamin Allen, the son of Williard Isaac Allen and Sarah Anne (Marsden) Allen, was born at Ithaca, N. Y., on August 31, 1883. He received his early education in the public schools of New York State. From 1908 to 1909 he was in the employ of the United States Government, in connection with pier and crib construction on Lake Ontario.

From 1909 to 1912, Mr. Allen was engaged as Tracer and also as Engineering Draftsman in the office of the State Engineer of New York, detailing and estimating in connection with Barge Canal bridges. From 1912 to 1914 he was connected with the Beaver Board Company, and also with the Shawmut Manufacturing Company, as Assistant Engineer in charge of design and construction, and also as Assistant to the Chief Engineer in connection with a 7 500-h. p. hydro-electric development and a steel and concrete pulp and paper mill costing $3 000 000.

From 1914 to 1915 Mr. Allen was Bridge Engineer in the office of the State Engineer of New York, in connection with the design of bridges over the Barge Canal. From 1915 to 1917 he was Assistant to the writer, and from May to August, 1917, he was at the Reserve Officers' Training Camp.

From August, 1917, to June, 1919, Mr. Allen was 1st Lieutenant with the 25th Engineer Regiment, U. S. A. He served in France from November, 1917, to May, 1919, receiving his discharge on June 2, 1919. Soon after his discharge from the Army he became connected with the Beaver Board Companies, Buffalo, N. Y., and was stationed at the plant at Anderson, Ind., as Engineer in charge of design and construction, where he remained until September, 1921. From September, 1921, until the time of his death, he was employed as Assistant to the writer.

Mr. Allen died very suddenly on June 17, 1922, at Albany, N. Y. He was married in Binghamton, N. Y., on December 20, 1906, to Marguerite May Finch, by whom he is survived.

Mr. Allen was devoted to his engineering work, in which he was intensely interested. He was of most pleasing personality and a thorough and conscientious workman. He had many devoted friends, by whom his untimely death is greatly regretted.

Mr. Allen was elected an Associate Member of the American Society of Civil Engineers on March 13, 1917.

* Memoir prepared by William Russell Davis, Assoc. M. Am. Soc. C. E.

JOHN EARL SHOEMAKER, Assoc. M. Am. Soc. C. E.*

DIED JUNE 22, 1922.

John Earl Shoemaker was born in Charleston, Ill., on September 26, 1881. He was educated at the University of Illinois, was graduated with the highest honors of his class, and received his A. B. degree in 1903.

His first engineering work was with the Wisconsin Bridge and Iron Company. From 1905 to 1907, he was connected with the U. S. Reclamation Service on various projects. From 1907 to 1909, he was with the Departamento de Obras Publicas, Cuba, and had charge of designing and constructing several large water-supply systems. From 1909 to 1912, he was employed as Designing and Construction Engineer for various irrigation projects in Colorado, Wyoming, and Montana.

Mr. Shoemaker came to Seattle, Wash., in November, 1912, and in January, 1913, accepted the position of Assistant Engineer with the Port of Seattle in charge of outside construction. In April, 1915, he was made Assistant Chief Engineer, and remained in that capacity until March, 1916, at which time he resigned to form a partnership with Peter Swensson, under the firm name of Swensson and Company, General Contractors. This partnership lasted six years, during which time work on many important contracts was done. He occupied the positions of President and Secretary, respectively, of the Seattle Master Builders' Association.

In the spring of 1922, after refusing a responsible position with the Asia Development Company, of Shanghai, China, Mr. Shoemaker accepted a position as Chief Engineer of a large coal mining company at Herrin, Ill. He was the first man killed in the cowardly and disgraceful riot at that place on June 22, 1922, while defending the Superintendent of the mine, a cripple, who was being mistreated by the strikers. He is survived by his widow and three small sons.

Mr. Shoemaker was a man of sterling qualities and charming disposition, and commanded the respect and esteem of all who knew him. He was an able engineer, noted for his tact and ability in handling men, and was an expert mathematician.

Mr. Shoemaker was elected an Associate Member of the American Society of Civil Engineers on June 6, 1911.

HILMAR FREDERICK SMITH, Assoc. M. Am. Soc. C. E.†

DIED JANUARY 6, 1923.

Hilmar Frederick Smith was born at New Brunswick, N. J., on January 13, 1892, his parents being the late Dr. John B. Smith, formerly Professor of Entomology in Rutgers College and State Entomologist of New Jersey,

* Memoir prepared by G. F. Nicholson, M. Am. Soc. C. E.

† Memoir prepared by B. F. Vandervoort, Assoc. M. Am. Soc. C. E.

and Marie von Meske Smith. He was educated at Rutgers College, New Brunswick, N. J., receiving the degree of B. Sc. in 1913, the degree of Electrical Engineer in 1920, and the degree of Civil Engineer in 1922.

Mr. Smith first began practicing his profession as a Student Apprentice with the General Electric Company, at Schenectady, N. Y., with which company he was engaged on the testing of electrical apparatus from September, 1913, to March, 1915. From March, 1915, to July, 1916, he was engaged as Assistant Engineer and later as Construction Superintendent with the Nixon Nitration Works, Millville, N. J., in connection with the construction of a $1 000 000 gun cotton plant. From July, 1916, to March, 1917, he was employed by Somerset County, New Jersey, as a Surveyor and Road Superintendent on macadam road construction. From March, 1917, to April, 1918, he was employed by the Pennsylvania Railroad on the New York Division as Transitman, on yard construction work at Princeton, N. J.

He entered the Government service in April, 1918, at U. S. Nitrate Plant No. 2, Muscle Shoals, Ala., holding at that plant the position of Inspector of Construction and Mechanical Superintendent, also acting as Technical Assistant to the Commanding Officer. In July, 1920, he was transferred from the Ordnance Department to the Construction Service, Quartermaster Corps, as Assistant Supervising Engineer, in charge of the Waco Quarry Project, Russellville, Ala., during which assignment he was in responsible charge of work amounting to $535 000. In September, 1921, he was transferred to Scott Field, Belleville, Ill., as Supervising Engineer for the Construction Service of the Quartermaster Corps of the Army, at which station he was in responsible charge of the engineering and inspection incident to the construction of a $1 300 000 airship hangar.

The work entrusted to Mr. Smith's supervision during his service with the Government was of considerably greater responsibility than that usually assigned to a man of his age, but his unusual ability along both theoretical and practical engineering lines, coupled with his exceptional devotion to his profession, enabled him to discharge his duties at all times in a manner highly satisfactory, and such as would have been a credit to a much older man than himself. As a man he possessed none but the most sterling characteristics, and was universally respected and admired by all his associates. His death is a distinct loss to the Engineering Profession, in which he was taking a foremost place for a man of his years, not only by carrying on successfully the important work on which he was engaged, but as a frequent contributor to the technical press.

Mr. Smith was elected an Associate Member of the American Society of Civil Engineers on November 25, 1918.